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PERFORMANCE REVIEW OF GRASS SWALE-PERFORATED PIPE STORMWATER DRAINAGE SYSTEMS

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RAC Project No. 585C

Report prepared by:

Paul Wisner and Associates Inc.

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PERFORMANCE REVIEW OF PERFORATED PIPE-GRASS SWALE STORMWATER DRAINAGE SYSTEM

FINAL REPORT (February 1993)

ABSTRACT

The research, reported in this study, addresses the Ministry of the Environment's research needs in the area of stormwater contaminant reduction techniques in urban stormwater runoff and the evaluation of groundwater recharge techniques.

The performance and potential of perforated pipes for stormwater quality control is evaluated. The studied design consists of a grass swale underlain by a section of perforated pipe enclosed in an exfiltration trench. The grass swale-perforated pipe system results in a pleasant curbless design and is often used in low density residential areas to replace open ditch systems, which perform poorly in winter-spring conditions.

The study conducts a systematic analysis and assessment of the performance of grass swales used in combination with perforated pipes through inquiries with municipalities, literature surveys, and laboratory and field testing. The study shows that such systems, when feasible, can result in a significant reduction of runoff quantity and hence pollutant loadings to receiving waters.

It was found that only a few cities in Ontario used grass swale-perforated pipe systems and have design guidelines for their installation. Currently employed practices were determined by: a) an inquiry with consulting system designers and pipe manufacturers, b) monitoring the installation of a typical system, and c) a thorough review of recent literature. Hydraulic laboratory and field tests were undertaken to relate the exfiltration flow rate to flow depth in the perforated pipe, pipe slope, surrounding soil characteristics, and perforation size and configuration. A three month field monitoring program was also conducted to gain a better understanding of the performance of perforated pipe systems under field conditions. In this field program, stormwater quantity and quality were monitored continuously for an entire summer season.

Based on the laboratory and field test results combined with the field monitoring program, a modelling approach was derived and implemented in a computer model which can serve as a tool for the design of grass swale-perforated pipe systems. General guidelines and recommendations on design aspects, construction, maintenance and applicability are also provided.

If found to be suitable for local conditions, grass swale-perforated pipe systems would not only provide an adequate stormwater drainage system, but would also provide a cost effective Best Management Practice (BMP) function for the management of stormwater quality.

EXECUTIVE SUMMARY AND CONCLUSIONS

Objectives:

Based on a research proposal prepared by Paul Wisner and Associates (PWA) funding was approved by the Ontario Ministry of the Environment to review and study the performance and potential of grass swale-perforated pipe systems in terms of stormwater quality and quantity control. The studied system consists of a grass swale underlain by a section of perforated pipe enclosed in a gravel trench. This system which results in a pleasant curbless design, is often implemented to replace open ditch systems which are known for their poor performance during winter-spring conditions. Typical ditch and grass swale-perforated pipe systems are shown on photos A and B in Figure I.

To assess the performance and potential use of grass swale-perforated systems the study undertook the following tasks:

1. Review of literature and municipal experience across Ontario.
2. Document currently employed design and construction practices for the installation of perforated pipe systems.
3. Analyze the hydraulic operation of grass swale-perforated pipe systems by means of hydraulic laboratory and field tests.
4. Monitor the runoff quality and quantity in two neighbourhood subdivisions with perforated pipe and conventional pipe systems (McFarlane and Amberwood, respectively) both in the Pine Glen area of the City of Nepean.
5. Develop a modelling procedure for the analysis and design of grass swale-perforated pipe drainage systems and incorporate the procedure into a computer program.
6. Derive general guidelines to improve the performance of grass swale-perforated pipe systems.

The location of the monitored perforated pipe and conventional systems are shown in Figure II. Results of additional monitoring done in another subdivision (Heart's Desire) were also used in this research for the calibration/validation of the model (point 5 above).

Findings:

The main findings of the study are summarized as follows:

Municipal Experience

The results of a survey conducted with 58 agencies (municipalities, towns, and cities) in Ontario having a population greater than 30,000, indicated that only 51% of the respondents have experience with perforated pipe systems. However, it was found that only few municipalities (around 20% of respondents) have some standards or guidelines for the design and installation of perforated pipes, grass swales, or infiltration trenches. These few existing guidelines were derived by the municipalities themselves based on experience and construction practices for conventional systems, which clearly shows the lack of standard practice for the design and installation of grass swale-perforated pipe systems in Ontario. A summary of the survey responses is provided in Table 1.

Hydraulic Laboratory and Field Tests

Through a series of laboratory tests, it was found that the flow through the perforations of the pipe which, in theory, can be computed with a simple orifice equation, is not directly proportional to the orifice area and/or to the square root of the pressure head created by the depth flow. The analysis of the results indicated that the orifice discharge coefficient varied with the head.

Field tests were conducted to measure the infiltration rates of typical grass swales and existing pipe trenches. It was found that infiltration rates were consistent with literature values.

Stormwater Quantity and Quality Monitoring

Results from measurements of numerous storms in a very wet season confirm that, for a perforated pipe with a gravel bed of 10 to 15 cm thickness, an equivalent initial rainfall abstraction (Ia) of 3 to 5 mm is representative.

The volumetric runoff coefficients were found to be 0.35 to 1.0 times the directly connected imperviousness ratio (Figure III) for Heart's Desire and McFarlane-Pine Glen, respectively. During the same period, the volumetric runoff coefficient for the traditional drainage system was significantly higher (Figure IV). The Rational Method peak flow runoff coefficient for events with a return period of less than two years was also reduced for the perforated pipe. The beneficial effect for the hydrologic budget of perforated pipes is shown in Figure V.

Average event concentration values from one event to the other were compared and there was no direct relation between their values and the antecedent number of dry days or the amount of rainfall. This variability was not significant for bacteria.

A direct comparison for the number of events monitored (7 to 11 per subwatershed) shows that average bacterial concentrations are somewhat higher in the perforated pipe systems. On the other hand, average concentrations of suspended solids, heavy metals and phosphorus are somewhat reduced. Because of the high variability it seem that for simulation purposes a statistical model may be appropriate for the analysis. This was demonstrated by a frequency analysis for E.Coli which shows that differences between the conventional and perforated pipe systems are within or close to the 90% confidence limit (Figure VI). While more data should be collected for the statistical analysis it is suggested as a practical conclusion that average concentration values may be the same. Because of the much lower runoff volumes in perforated pipe systems, they produce much lower pollutant loadings as illustrated in Table II, and in Figure VII for E.Coli only.

The quality control performance of perforated pipes can be further improved by increasing the depth or width of the gravel bed to increase storage and enhance infiltration. On the other hand, monitoring at McFarlane shows that if the groundwater table is high and a base flow is maintained during dry periods, the hydraulic performance can be drastically reduced. Hydrogeologic studies prior to installation of perforated pipes are, therefore, of essence.

Modelling

A computer model was developed to serve as a tool for the design of grass swale-perforated pipe systems. The model, which was calibrated and validated using experimental as well as field data, performs detailed computations for flow through the system on a lot by lot basis (i.e. from one catchbasin to another). Several parameters affecting the system performance are considered in the modelling approach. These included especially lot size and imperviousness, grass swale dimensions and its infiltration capacity, pipe length, number of orifices and their configuration, trench dimensions and native soil infiltration capacity.

The model was used to simulate the minimum trench depth required to capture runoff from a 25 mm storm for different native soils and different lot imperviousness ratios. Results of these simulations are shown in Figure VIII. Trench depths varied from 0.3 to 1.4 m depending on native soil infiltration capacity and lot imperviousness.

The model was also applied to an existing system in Heart's Desire subdivision in Nepean, Ontario. Based on a single lot analysis, the simulated overflow volume was used to compute a volumetric runoff coefficient. This latter, which was in good agreement with the observed value, was used to simulate an overflow hydrograph for the entire watershed using the OTTHYMO-89 model. Although the OTTHYMO simulated hydrograph was computed with a constant volumetric runoff coefficient (as compared to a variable coefficient used by the detailed model, the results were found to be in good agreement with the observed hydrograph (refer to Figure IX).

General Conclusions

While many municipalities utilize swales and perforated pipes for urban drainage systems, literature data on their hydraulic, hydrologic and runoff quality performance was scarce and inconclusive.

The field monitoring conducted in this study shows that runoff volume reduction for frequency flows is significant. Measurements of pollution concentrations are not as conclusive, but the effect of perforated pipes on reduction of pollutant loading seems to be considerable. The efficiency of the perforated pipe as a BMP can be improved by adequate design. This design and consideration of different pipe orifices or soil conditions can be analyzed by means of the model developed in this research.

To assure a good performance, however, it is necessary to locate the storage above the groundwater table and to assure sediment control during construction. Other stringent design aspects are discussed in the report.

Table I: Summary of Questionnaire Results
(Based on 40 responses from Ontario, 8 from Western Canada and 5 from the USA)

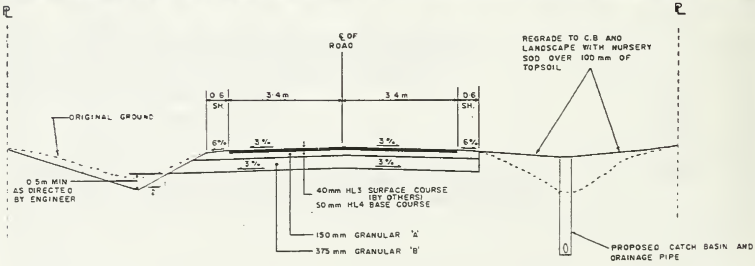
Survey Question	Affirmative
Organizations who used perforated pipe systems in the past.	51%
Type of pipe used:	
Corrugated steel pipe	59%
Corrugated polyethylene perforated pipe	44%
Smooth-wall polyethylene perforated pipe	22%
PVC pipe	37%
Other (clay pipe)	4%
Application:	
Storm sewer system with grass swales	52%
Stormwater distribution for infiltration in trench	52%
Other (Road and embankment subdrain, septic field, leachate collection)	52%
Design criteria available for installing perforated pipes for urban runoff infiltration.	19%
Return period of storm used to size the pipes (when used as storm sewers)	36%
2 year	43%
5 year	36%
Other (10 year, up to 100 year, and more)	
When filter fabric is used, where is it placed:	
Around perforated pipe	72%
Around entire trench	64%
None	7%
Willingness to consider using perforated pipe system as an alternative to other methods of stormwater management.	76%

Table II: Pollutant Loadings from Perforated and Conventional Pipe Systems during the Period of June 15 to Sept. 20, 1992

Parameter	Unit	Base flow INCLUDED		Based flow EXCLUDED	
		Perforated Pipe	Conventional Pipe	Perforated Pipe	Conventional Pipe
		McFarlane	Amberwood	McFarlane	Amberwood
NO_3-N	kg/ha	1.18	1.90	0.26	---
TKN	kg/ha	0.37	1.19	0.33	1.09
PO_4	kg/ha	0.25	0.79	0.16	0.65
TSS	kg/ha	24	235	22	232
Cl	kg/ha	96	21	23	---
Cu	kg/ha	0.0028	0.01100	0.00270	0.01100
Pb	kg/ha	0.00023	0.00340	0.00023	0.00340
Zn	kg/ha	0.026	0.110	0.022	0.097

Photo A: Ditch System

Photo B: Grass Swale-Perforated Pipe System



Cross-Section of Ditch and Grass Swale-Perforated Pipe Systems

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FIGURE I. CONVENTIONAL AND GRASS SWALE-PERFORATED PIPE SYSTEMS

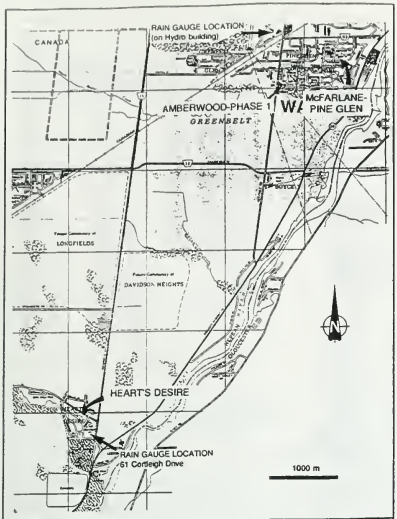

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FIGURE II LOCATION OF THE PERFORATED AND CONVENTIONAL SYSTEMS MONITORED BY PWA

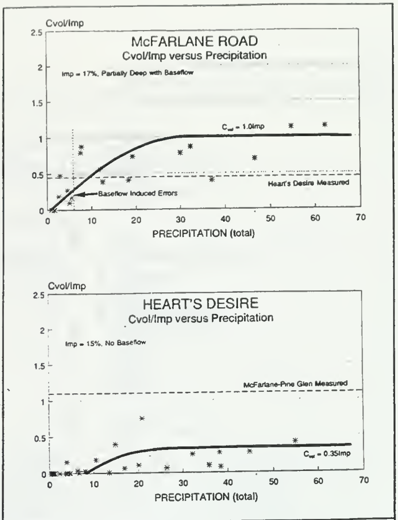

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FIGURE III PLOT OF C_{vol}/IMP VERSUS P FOR McFARLANE-PINE GLEN AND HEART'S DESIRE

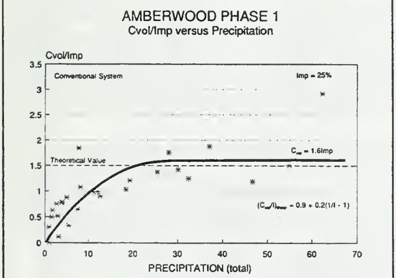

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FIGURE IV PLOT OF C_{vol}/IMP VERSUS P FOR AMBERWOOD PHASE 1

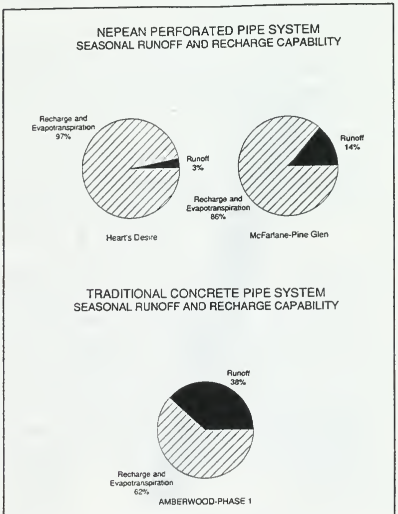

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FIGURE V SEASONAL RUNOFF AND RECHARGE CAPABILITY EXPRESSED AS A PERCENTAGE OF TOTAL RAINFALL

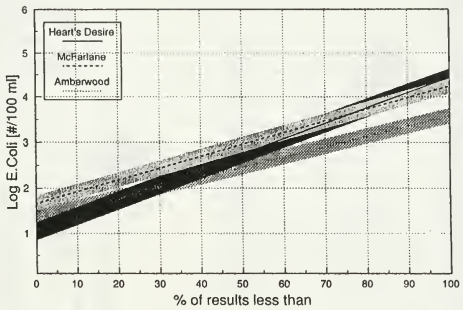

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FIGURE VI 90% CONFIDENCE INTERVALS (E. COLI)

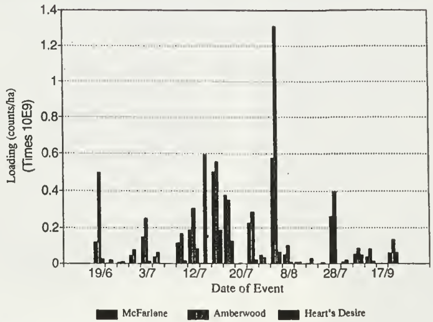

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FIGURE VII EVENT E. COLI. LOADINGS

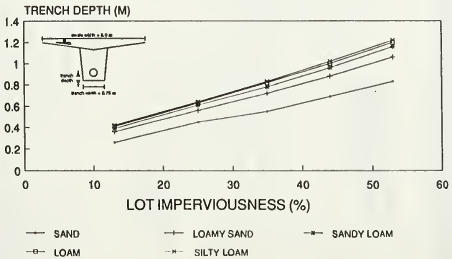

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FIGURE VIII MINIMUM TRENCH DEPTHS REQUIRED TO CAPTURE RUNOFF FROM THE ONE-INCH CHICAGO STORM (TOP SOIL = LOAM).
(see Section 6.1 for other swale, pipe and trench data)

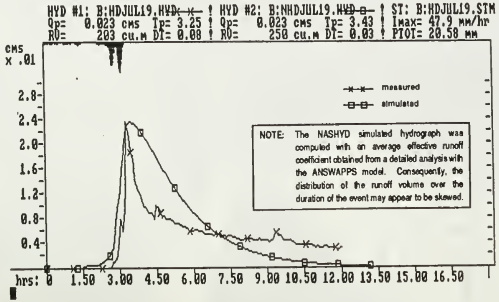

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FIGURE IX COMPARISON BETWEEN OBSERVED AND SIMULATED OVERFLOW HYDROGRAPHS (HEART'S DESIRE SEWERSHED, EVENT OF JULY 19, 1992).

PERFORATED PIPE-GRASS SWALE STORMWATER DRAINAGE SYSTEM

PROJECT NO. 585C

FINAL REPORT
(November 1992)

1.0 INTRODUCTION

While in the past, municipal pollution to receiving waters was related mainly to wastewater systems, more recently surface water runoff has been identified as a major cause of pollution. A variety of pollutants accumulate within an urban area and are subsequently carried by stormwater runoff to rivers and lakes. This results in impaired water use for purposes such as water consumption, groundwater recharge, and recreational activities.

Early stormwater management practices concentrated on reducing peak post-development runoff levels to minimize downstream flooding caused by urbanization. This was typically accomplished by constructing stormwater detention ponds, which were designed as dry systems that would eventually discharge the entire detained volume of runoff to receiving waters. Therefore, such detention ponds just redistribute the rate of runoff over a period of time and do not reduce the total stormwater volume. More recently, and in response to growing concerns over the contaminating impacts of urban runoff on receiving waters, stormwater management alternatives started to address the problem of water quality control. Recharge and mitigation of changes in the hydrologic budget also became prime objectives.

Conventional Best Management Practices (BMP) for runoff quantity and quality control now include wet ponds, wetlands, vegetative filters, and various infiltration practices. Recently, attention has focused on structural measures for water quality improvement through non-conventional BMP's such as ultraviolet (UV) light treatment, which proved to be effective for bacterial reduction. However, these practices may be, in some conditions, relatively expensive for widespread use. As an alternative to on-line storage basins, infiltration practices, including off-line retention basins, swales, infiltration trenches and pervious pavements are being increasingly considered in the US and Canada as an alternative means of quantity and quality control.

In this study, the performance and potential of perforated pipes for stormwater quality and quantity control is evaluated. The studied design consists of a grass swale underlain by a section of perforated pipe enclosed in an exfiltration trench. This system which results in a pleasant curbless design, is often implemented to replace open ditch systems known for their poor performance during winter-spring conditions. Typical ditch and grass swale-perforated pipe systems are shown on photos A and B in Figure 1. To assess the performance and potential use of grass swales-perforated pipe systems a number of tasks were undertaken.

1. Evaluate past experience and currently employed design practices in Ontario for the installation of perforated pipe swale systems by: (a) conducting a thorough review of recent literature (Section 2), (b) conducting an inquiry with pipe manufacturers (Section 3) and (c) monitoring the installation of a typical system (Section 4)

2. Based on theory and literature a modelling approach for the hydrologic and hydraulic analysis of grass swale-perforated pipe systems was developed (Section 5). The approach combines related aspects of the grass swale characteristics with the perforated pipe characteristics and also accounts for the effects of soil infiltration capacity, trench dimensions, pipe perforation dimensions and density, etc.

3. Hydraulic laboratory tests were undertaken to relate the exfiltration flow rate out of the pipe, to flow depth, pipe slope, and perforation size and configuration (Section 6.1). Other tests, conducted in the field were performed to estimate infiltration rates in grass swales and trenches (Section 6.2).

4. Based on the derived modelling approach a computer model to serve as a tool for the design of grass swale-perforated pipe systems was developed. The model is calibrated and validated using the experimental as well as field data (Sections 7 and 8).

5. The stormwater quality and quantity management capabilities of existing grass swale-perforated pipe systems were investigated in the field and compared with the performance of a conventional concrete sewer pipe system. A detailed monitoring program, in which rainfall and runoff quality and quantity were continuously measured was undertaken over a complete summer season at two perforated pipe systems and at one conventional concrete sewer pipe system (Section 9 for stormwater quantity and Section 10.1 for quality).

6. In order to investigate the effects of perforated pipes on groundwater level and quality, a monitoring program was undertaken during the spring snowmelt at a newly installed system (Section 10.2).

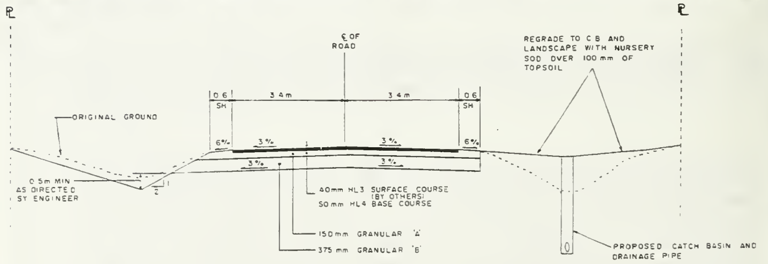
7. Based on findings, guidelines and recommendations on applicability, construction and maintenance of grass swale-perforated pipe system are provided (Section 11).

The study shows that, when feasible, drainage systems which incorporate the use of grass swales and perforated pipes can significantly reduce runoff quantity and pollutant loadings to receiving waters.

The joint use of such system with other Best Management Practices such as quality ponds could not only lead to smaller detention facilities but could also enhance their performance.


Photo A: Ditch System


Photo B: Grass Swale-Perforated Pipe System



Cross-Section of Ditch and Grass Swale-Perforated Pipe Systems

PAUL WISNER & ASSOCIATES INC. Water Resources and Environmental Consultants Ottawa · Toronto	FIGURE I. CONVENTIONAL AND GRASS SWALE-PERFORATED PIPE SYSTEMS

2.0 LITERATURE REVIEW

The combined use of grass swale-perforated pipe systems for urban runoff infiltration is lightly covered in the literature as the idea is relatively new.

This chapter presents a review of literature not only on the combined use of perforated pipes and grass swales but also on other related subjects. These include the use of grass swales for quantity and quality control, the use of infiltration trenches for management of runoff, the use of perforated pipes for drainage of groundwater, and the use of perforated pipes in leaching fields.

2.1 Stormwater Runoff and Its Quality

Urbanization has been recognized as responsible for the increase in both peak flow and total volume of surface runoff because of the decrease in infiltration as compared to pre-developed conditions. Urban stormwater has also been identified as a major contributor to the pollution of water courses as it typically contains high suspended solids and bacteria levels (Marsalek 1986). Suspended solids are detrimental not only because they increase water turbidity but also because other harmful pollutants such as metals, nutrients, and pesticides are adsorbed onto their surface. The principal pathogens present in stormwater runoff include bacterial pathogenic organisms, viruses, fungi, protozoa and algae.

The concentration of pollutants in stormwater runoff is generally higher at the beginning of a storm and then decreases as runoff continues (Livingston 1988). This early runoff with high pollutant loads is typically referred to as the first flush. Shaver (1986) indicated that the first flush is considered to be 0.5 in of runoff per impervious area. Studies in Florida also indicated that, for a variety of land uses, the first 0.5 in of runoff contained 80-95 % of the total annual loading of most stormwater pollutants (Livingston 1988). Therefore, diverting the first flush of runoff from receiving waters results in the removal of the majority of total annual pollutant loadings.

Small rainfall events, which carry the highest pollutant concentrations, are frequent in most parts of Ontario. A study for the Rideau River (1983) stated that 93% of the rainfall events generated in the Ottawa region during the summer months (June-August) have less than 25 mm of total rainfall. Measures for pollution control for this type of rainfall events are essential.

Water quality improvement can be accomplished through the use of Best Management Practices including dry/wet ponds, wetlands, vegetative filters, and various infiltration practices. Wet ponds are in wide spread use but are coming under increased scrutiny as temperature effects and long term maintenance aspects

are a concern to the municipalities responsible for maintaining them. As an option to on-line storage basins, infiltration techniques (off-line retention basins, swales, infiltration trenches and pervious pavement) are being considered increasingly in the US and Canada as means of both quantity and quality control.

2.2 General Infiltration Practices

A number of references describe the design techniques for infiltration trenches (Goddard, 1984; Paul Theil Associates Limited, 1980; Schueler, 1987; Stahre and Urbonas, 1989). Stahre and Urbonas (1989) provided a design method for infiltration and percolation facilities. Their method uses the Rational Method formula to calculate the volume of water entering the basin. They recommended this volume be increased by 25 percent to account for the rainfall before and after the most intense part of a rainstorm.

Stahre and Urbonas do not recommend the use of percolation for stormwater disposal when:

1. seasonally high groundwater is less than 2 feet (0.6 m) below the infiltrating surfaces;

2. bedrock or an impervious soil layer is within 4 feet of the infiltrating surface;

3. percolation bed is located in fill;

4. the adjacent and underlying soils are classified by SCS as Hydrologic Group C or D, or the saturated hydraulic conductivity is less than 2×10^{-5} m/s.

The Japanese used on-site detention facilities to store stormwater (Nakamura 1988). Storing stormwater on-site not only controls stormwater discharge, but also makes use of the water instead of just wasting it. Use of stormwater for flushing toilets is one of the typical ways of usage. In this context stormwater is viewed as a resource to be managed and used in support of societal benefits, as opposed to the traditional approach, where it is considered as a waste to be drained and disposed of as quickly as possible.

The Japanese have also made extensive use of infiltration trenches and perforated pipes for over a decade. The Tokyo Experimental sewer System combines a novel catchbasin design along with a specially designed curb to allow inflow to an infiltration trench. The system has been tested extensively and has been found to reduce suspended solids loadings by 53%.

Fujta (1987) described experimental sewer systems (ESS) and their application in Japan. ESS has been developed and practised in densely populated districts in Tokyo and was found to be effective for both quantity and quality control. ESS mainly consists of infiltration facilities and storage facilities. The infiltration facility unit include infiltration inlets and trenches placed along both sides of roads and a permeable pavement, applied to all of the roads running along sewer mains. Storage is included inside infiltration facilities, circuitous route of sewer pipes and in manholes where a weir is installed to store the peak flow in the upstream trunk sewer pipes. Infiltration facilities were observed to perform well even after clogging has occurred as it takes a long time before the facility is completely clogged. Various anti-clogging devices such as buckets, baskets, mudpits, porous plates and screens were considered.

The Swedish Association of water sewage Works (1983) presented recommendations to determine whether infiltration facilities are feasible at a particular site. This evaluation system, however, is intended for preliminary screening of potential sites and should not be considered as a substitute for sound engineering and site specific investigation. If a site happens to be feasible for infiltration facilities the Swedish Association recommended guidelines for the design of such facility. It is recommended to reduce field conductivity values by a safety factor of 2 or 3 for design to account for soil clogging with time. It is also recommended to assume that all water percolates into the ground only through the sides of the basin as the bottom is more susceptible to clogging with time.

A major potential problem with any infiltration practices is the accumulation of sediments (which may contain elevated levels of metals [lead, copper, zinc, nickel], nutrients [phosphorous and nitrogen], BOD, COD, and Bacteria) which result in clogging of the infiltration pores.

Harrington (1989) indicated in a survey of 200 infiltration installations in Maryland, that of the 67 sites that were not functioning as designed, approximately 70% showed signs that sediment had entered the system either during or after construction. In Montgomery County, Maryland, any of the infiltration facilities built during the 1970's have failed. The reason for failure is primarily due to inadequate soil investigation, lack of sediment control, absence of vegetated buffer strips and poor construction techniques. Harrington (1989) recommended mitigative measures to alleviate the clogging problems. He suggested that trenches should be constructed only after the entire site is stabilized and to provide a minimum of 2.0 ft of effective vegetated buffer strip after construction to minimize the accumulation of sediment over the infiltration surface.

Schueler (1987) provided detailed design for infiltration trenches. Table 1 contains the estimated long-term pollutant removal rate for water quality trenches.

Table 1: Estimated Long-Term Pollutant Removal Rates From Water Quality Trenches

POLLUTANT	DESIGN VOLUME BASED ON	
	0.5" (12.5 mm) Runoff From Impervious Area	Runoff From 1"(25 mm) Storm
SEDIMENT	75%	90%
TOTAL PHOSPHORUS	50-55%	60-70%
TOTAL NITROGEN	45-55%	55-60%
TRACE METALS	75-80%	85-90%
BOD	70%	80%
BACTERIA	75%	90%

Nightingale (1989) studied the chemical quality of soils, water percolating through soils as well as groundwater mounding underneath five retention/recharge basins receiving urban stormwater runoff. The study showed that the concentrations of copper (Cu), iron (Fe), silver (Ag), nickel (Ni) and lead (Pb) in the recharge-mound water were similar to the regional groundwater values and the surface few centimetres of the basin soils have accumulated As, Ni, Cu and especially Pb.

Yousef, et al (1986) summarized existing design considerations and removal efficiencies of selected pollutants in retention systems. A study was conducted on a retention pond, located at the Maitland Interchange on Interstate 4, north of Orlando, Florida. Part of the study included groundwater monitoring and it was determined that there is no evidence to indicate that metals concentrated in the sediments of the pond are migrating. Given this, it is very unlikely that a pollution hazard exists to nearby surface or groundwater. However, it should be noted that this may also depend on the type of natural soil.

Kuo et al (1989) developed a two-dimensional finite element model to simulate the transient flow in a variable saturated porous medium for the study of infiltration trenches. Routing is performed to find infiltration rate, water depth and storage in the trench based on parameters such as soil properties, water table location, initial soil moisture conditions, trench geometry, and surface runoff hydrograph at the facility site. An experimental program was also conducted to test the validity of the finite element model results.

According to this study the infiltration rate was observed to be higher in a narrow, deep trench than in a wide shallow one because the deeper the water the higher the pressure head. Conversely, a wide, shallow trench allows a greater volume of water to infiltrate because of the large horizontal area through which vertical infiltration takes place. The depth of the water table has been shown to have a greater effect in silt than in sand. There exists a limit for each soil beyond which the water table has no effect on infiltration. The limit for sand was found to be about 10 ft (3 m), 30 ft (9 m) for loamy sand, and 60 ft (18 m) for sandy loam. The developed finite element model is considered as a useful design tool for infiltration facilities especially because the formulation has the capability to change geometries and to calculate the flux and water level in the trench, which are important factors in sizing the trench.

Graham (1990) developed a two-dimensional finite element saturated-unsaturated flow model to analyze the performance of an infiltration basin under transient flow conditions. The model, which was coupled with a surface routing model, was used to analyze the response of the underlying soil environment to stormwater runoff inputs into the infiltration basin. Simulations were performed to study the variations in seepage and pollutant transport with changes in the maximum ponding depth, the antecedent soil moisture conditions, the location of the water table, and the textural characteristics of the soil. The main findings of this study are summarized as follows:

1. The infiltration rate is significantly affected by the location of the water table. There is a critical distance to the water table below the basin at which groundwater mounding causes significant increase in the drainage time.

2. The soil textural class in an important parameter that must be carefully considered during the design stage of the basin.

3. The model developed can be successfully used to analyze the impact of urban infiltration basins on the groundwater environment.

Duchene (1991) simulated infiltration of stormwater runoff in an infiltration trench using a two-dimensional saturated-unsaturated finite element model. Changes in infiltration rate with respect to time, depth of water in the trench, distance to the water table, surrounding soil texture and antecedent moisture condition were investigated. Results obtained by the finite element model were compared to those obtained using a much simpler approach based on Darcy's law. Infiltration rates computed by this latter method were shown to be more on the conservative side.

2.3 Grass Swales

Grass swales provide some stormwater quantity and quality management for small design storms by infiltration and flow attenuation. Field studies and modelling efforts indicate that swales can filter out particulate pollutants (Schueler, 1987). Swales attenuate and reduce the peak discharge. The grass reduces the runoff velocity thus increasing the watershed time of concentration and attenuating the peak discharge. Grass swales also infiltrate a portion of the runoff which reduces the total runoff volume and peak discharge.

Pollutants are removed in grass swales by the filtering action of the grass, deposition in low velocity areas, and by infiltration of runoff into the sub soil. Field monitoring has provided mixed results as to the extent of pollutant removal achieved by grass swales. The pollutant removal of swales depends on the longitudinal slope, side slope, type of cover and underlying soils. Moderate removal of particulate pollutants can be expected during small storms if swale slopes are graded as close to zero as drainage will permit, side slopes are no greater than 3:1, a dense cover of grass is provided and the underlying soils have a high permeability. (Schueler, 1987).

Wanielista et al (1986) developed a model which can be used to calculate an average infiltration rate for a grass swale. The equation considers the length of swale, the side slope, the longitudinal slope, Manning's roughness coefficient and the infiltration rate of the swale. The equation, which is based on mass balance, was developed for triangular shaped swales underlain by sandy soils.

2.4 Perforated Drains/Perforated Pipes

Numerous studies have been conducted on perforated drains *(Mohammed and Skaggs, 1983; Broughton et al, 1986 among others)*. Mohammed and Skaggs (1983) conducted laboratory tests to determine the effects of total opening area, location of openings and gravel envelope on radial flow to 100 mm diameter corrugated plastic drains. Radial flow theory was used to evaluate the effective drain tube radius and the drainage transfer coefficient for the corrugated tubes. The effective drain radius is the radius of a completely open tube such that the flow to this open tube is the same as the flow into the actual tube. The drainage transfer coefficient is an entrance resistance used for calculating the head loss at the drain tube wall. The use of this perforated drain theory can be applied to study the performance of perforated pipe for draining of the subsurface soils after a rainfall.

Goddard (1984) reported in his paper the successful utilization of underground drainage systems to solve stormwater disposal problems in Titusville, Florida. Titusville installed perforated drainage pipes at a number of locations and felt that this was the best method, in its experience, for dealing with stormwater disposal and

storage. The location of the water table was found to be a very important consideration in the application of this method of underground drainage. The system was proved to work best where the water table rises to a maximum height of 30 to 36 in (75 to 90 cm) below ground surface or deeper.

In a study conducted by Delcan Corporation in 1988, the performance of perforated corrugated steel pipes installed in various residential areas of the City of Ottawa over the past 10 years, was assessed. The assessment involved monitoring and testing at 10 sites, which were drained by conventional sewers, ditches or perforated corrugated steel pipes, over a three year period from 1985 to 1987. The monitoring program especially included measurements of flows and collection of grab samples which were done manually for several large storms. Rainfall characteristics were described as related to raingauges outside the monitored system and the timing of the sampling was not related to a given rainfall intensity. Delcan recommended, however, future monitoring to be done continuously and by automatic samples.

The main findings of Delcan's study were that flows from local perforated pipes were reduced compared to those in conventional storm sewers and local ditches, the groundwater recharge was increased and the pollutant levels were decreased. Based on the results of a questionnaire the same study showed that perforated pipe systems are more acceptable to homeowners than ditch systems. The negative aspects revealed in the monitoring program mainly included a slight rise in bacterial pollutant concentrations as well as phosphorous levels in the perforated pipe systems compared to local conventional storm sewers and also root intrusion in some of the perforated pipes indicating increased special maintenance costs and perhaps shorter system life.

Measured peak flows showed reduced values compared to traditional Rational Method flows. These reductions were found to vary with return period, drainage basins, and native soils. Volumetric runoff coefficients could not be accurately determined due to the nature of the measurement method which was used. The Delcan study gives therefore, only qualitative assessment of perforated pipe hydraulics and hydrology. However, it gives very useful information on public opinions in areas with perforated pipes, design practices and overall impact if the perforated pipe is used in conjunction with a treatment unit.

A recent study completed by Paul Wisner and Associates (PWA, 1990) studied the as-built performance of a grass swale perforated pipe system in a local subdivision. Although the swales were effective in infiltrating the first few centimetres of rainfall, performance tests on the perforated pipe indicated that exfiltration rates from the pipe were extremely small. It was observed that fine sediments had clogged perforations in the pipe, the consequence of probably poor construction practices.

Duchene (1991) conducted an experimental program and studied the hydraulics of orifices in perforated pipes. He found that the steady state discharge rate through the orifices of the pipe can be determined with the orifice equation. An extension of his work would be to determine the effect of different orifice configurations, and the use of baffles to increase flow depth in the pipe and exfiltration rates. The hydraulic performance of perforated pipes should also be compared to that of conventional storm sewers.

Based on this thorough review of the literature, it was shown that while much work was done with respect to infiltration practices, the performance of grass swale-perforated pipe systems is still not well understood. Research is required to gain better understanding of the hydraulics of perforated pipes. The effect of grass swales and top soils in terms of not only infiltrating stormwater but also enhancing its quality needs also to be evaluated. Last, but not least, the impact of the proposed system on stormwater as well as groundwater quality needs to be determined.

3.0 SURVEY OF CURRENT DESIGN AND INSTALLATION PRACTICES

A survey, in the form of a questionnaire, was used to determine current design practices and construction methods for perforated pipe systems. The questionnaire was sent to 62 agencies (municipalities, towns, cities and consultants) in Ontario with a population exceeding 30,000. Additional questionnaires were also distributed in British Columbia and Alberta. A total of 53 completed questionnaires have been returned. A copy of the questionnaire is shown in Appendix A and the general questionnaire results are summarized in Table 2.

3.1 Experience with the System:

51% of the respondents have experience with perforated pipes systems. The City of Nepean, the District of North Vancouver, and the District of Saanich in British Columbia have applied perforated pipe systems in more than 10 stormwater projects. The main types of perforated pipes used were made of corrugated steel and polyethylene; some PVC, and clay pipes were also used. 76% of respondents are willing to use perforated pipe systems. This percentage includes previous users as well as those willing to apply the system for the first time.

3.2 Types of Application:

The different types of perforated pipe systems were found to be equally used as storm sewers, infiltration distribution systems, road and embankment subdrains, in septic field distribution, and leachate collection in a landfill site. Although several respondents used grass swales for conveying surface flows, only a few municipalities used a combined system of grass swales and perforated pipes for the purpose of urban runoff infiltration.

Table 2: Summary of Questionnaire Results
(Based on 40 responses from Ontario, 8 from Western Canada and 5 from the USA)

Survey Question	Affirmative
Organizations who used perforated pipe systems in the past.	51%
Type of pipe used:	
Corrugated steel pipe	59%
Corrugated polyethylene perforated pipe	44%
Smooth-wall polyethylene perforated pipe	22%
PVC pipe	37%
Other (clay pipe)	4%
Application:	
Storm sewer system with grass swales	52%
Stormwater distribution for infiltration in trench	52%
Other (Road and embankment subdrain, septic field, leachate collection)	52%
Design criteria available for installing perforated pipes for urban runoff infiltration.	19%
Return period of storm used to size the pipes (when used as	
storm sewers)	36%
2 year	43%
5 year	36%
Other (10 year, up to 100 year, and more)	
When filter fabric is used, where is it placed:	
Around perforated pipe	72%
Around entire trench	64%
None	7%
Willingness to consider using perforated pipe system as an alternative to other methods of stormwater management.	76%

3.3 Design Standards

Only 19% of respondents (10 organizations) have some design criteria or guidelines for the design and installation of perforated pipes. Similarly, 19% and 22% of respondents have design and installation guidelines for infiltration trenches and grass swales respectively.

Several cities in Ontario (Guelph, Ottawa and Nepean) provided design standards for perforated pipe/swale systems. These design guidelines are shown in Appendix B.

The standards cover many items including: pipe sizes, trenches, catchbasins, manholes, top soils, weeping tile drainage considerations, maintenance and monitoring. The 2 year design storm is generally used to size the perforated pipes when used as part of a storm drainage system. The minimum pipe sizes recommended by the Cities of Ottawa and Nepean are 200 and 250 mm respectively. In conjunction with storm sewer systems, corrugated steel and smoothed wall polyethylene perforated corrugated pipes are recommended.

The standards indicate that if root intrusion is a potential problem it can be alleviated with copper or copper sulphate if polyethylene pipes are used. Alternatively, the use of solid non-perforated pipes is recommended in extensively treed areas of the system. The use of copper wires within the trench or a copper mesh around the pipe has also been considered to minimize root intrusion.

Mitigative measures to control clogging of pipe perforations as well as trench sides are also recommended. These include not only placing filter socks around the pipe and geotextiles around the trench, but also the provision of a prefabricated corrugated steel pipe tee or elbow as catchbasin. Such a design allows the perforated pipe to continuously traverse the catchbasin.

The minimum pipe bedding required by the City of Nepean is 75 mm. However, the City of Ottawa recommends to increase this depth to 300 mm to provide greater storage, increase groundwater recharge potential, increase flow along bedding and extend system life. Manholes and catchbasins are provided in accordance with conventional sewer requirements with a minimum 300 mm sump depth. Manholes are located at pipe junctions, and spaced at intervals suitable for maintenance purposes. Pipe installation is in accordance with conventional sewer construction requirements for proper pipe bedding and backfilling.

3.4 Pipe Sizing

Pipe sizing criteria varies with their intended use. For urban runoff applications, systems are designed to convey the runoff from a two or a five year storm. Respondents from British Columbia use the ten year storm to size their pipe systems.

3.5 Trench Sizing

The trench sizing criteria includes minimum bedding below the pipe, minimum cover above the pipe obvert, minimum width, and type of backfill. Typical values for trench

dimensions are illustrated in Figure 2. Minimum bedding ranges from 0 to 300 mm with an average of 150 mm. With the exception of the City of Ottawa, it is not clear whether bedding criteria (depth and width) considers storage. Most bedding criteria likely accounts for structural requirements related to pipe support.

Minimum cover above the pipe ranges from 100 to 900 mm with an average of 400 mm. One respondent (City of Scarborough) uses the Provincial Standard for flexible sanitary sewers for specifying the bedding. This standard requires a minimum cover, c, equal to:

c	=	300 mm	for diam. < 600 mm
c	=	Diam./4 + 300	for diam. >= 600 mm

3.6 Backfill Material

The backfill material used in trenches is either rock, stone, or sand. Most users specify clear stones (uniform grading) and clean conditions for the backfill material. Backfill material size ranges from 19 to 50 mm with an average of about 30 mm.

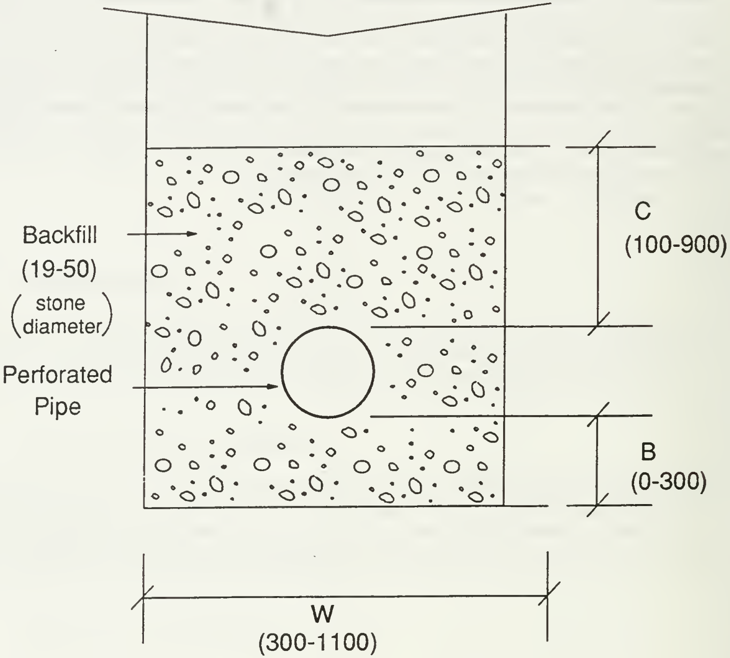
3.7 Protection Against Clogging

More than two thirds of respondents place filter fabric around perforated pipes and also around the entire trench to prevent the migration of fine sediments that could create clogging. Other protective measures during construction include closing open pipe ends (before connecting), maintaining catchbasins closed, and the use of silt traps made of clear stone and filter fabric.

3.8 Evaluation and Monitoring

Most respondents did not answer this part of the questionnaire. Therefore, it was assumed that no monitoring or evaluation have been performed. Visual inspections were reported in three occasions. These inspections especially involved infrequent visual observations of surface water ponding in the sump pit of catchbasins. Some of the evaluation tests indicated that pipes were occasionally broken accidentally while performing maintenance on other utilities. The City of Nepean conducted a monitoring and sample testing program in September, 1991, on a Perforated Pipe/Swale System in the Pine-Glen Subdivision, however, due to instrument failures no data was made available.

The City of Oshawa (1976) conducted a study of various underdrain pipe systems to evaluate the effectiveness of underdrains used in road subgrade structures. Major problems, that might affect the efficiency of the system were discovered. These included mainly: (i) infiltration of material into the pipes causing partial or complete blocking, (ii) shearing or breaking of the underdrains at catchbasins, and (iii) the opening up of the coupling during the placing of the filter media. In order to minimize or eliminate some of these problems the City of Oshawa conducted laboratory and field tests to further assess the system performance. Based on this study, the City recommended the use of corrugated plastic tubing rather than smooth wall plastic pipes. Furthermore, all joint couplings must be manufactured so that a 4 inch overlap on each side of the joint is achieved.



Dimensions in mm.

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FIGURE 2. RANGE OF TRENCH DIMENSIONS USED BY OTHERS

4.0 INSTALLATION OF A GRASS SWALE-PERFORATED PIPE SYSTEM

The installation of a grass swale-perforated pipe was observed during several visits to the selected site. Photographs of the installation process are included in Appendix C.

4.1 Location

After discussions with the MOE district office, a site in the City of Nepean, draining into the Rideau River, was selected for monitoring the installation of swale-perforated pipe system. Figure 3 shows the location of the site. The City selected a section in the Pine Glen area for drainage improvements and decided to install a smooth wall polyethylene perforated pipe system combined with shallow grass swales to replace the existing ditch/swale drainage. As illustrated in Figure 4, installation included two street segments: a) a portion of Promenade Avenue and b) Sheridan Avenue.

4.2 Installation Procedures

4.2.1 Excavation

Trenches were excavated using a backhoe at a constant width of about 0.7 m. Trench depths ranged from 2.0 to 3.2 m below the existing ground. The depths were controlled using a sight level and surveyed bench marks as well as a laser grade line level placed along the centreline of the ditch. The excavated soil was piled alongside the trenches to be used later for partial backfilling. Asphalt surfaces at road crossings and driveways were cut using a concrete saw prior to excavation.

4.2.2 Liner, Gravel and Pipe Segments

Following excavation, the sides of the trenches were lined with Terrafix 360R non-woven geotextile filter cloth. The cloth was held in place by tying the ends to the top of the trench walls. Excess filter cloth in shallower trench locations was used to line the bottom of the trench.

Up to 100 mm of coarse gravel was placed at the bottom of the trench for bedding. The bedding was also used for final adjustment of the pipe elevation resulting in a variable bedding thickness. The pipe system was then laid down by hand over the bedding. Pipe sizes ranged from 300 to 450 mm diameters. Pipe segments were joined using overlapping plastic couplings secured with plastic locking pull ties. Filter socks coverings were overlapped over the joints and provided continuous coverage. In order to provide additional pipe strength, sections of solid PVC pipe

were installed under the private driveways. These solid sections extended at least one metre beyond the edge of the driveways.

The trench was backfilled with 25 mm clear stones to a maximum depth of about 0.8 m. Between 200 and 300 mm depth of fine gravel was placed on top of the larger, 25 mm stones. The Terrafix filter cloth was then overlapped at the top of the trench. The remaining depth over the filter cloth was backfilled with the native soil piled on the side from excavation.

4.2.3 Catchbasin and Manhole Installations

A least one catchbasin or manhole was installed between any two driveways. Prefabricated Corrugated Steel Pipe (CSP) 'tee' catchbasins were installed at the site. As illustrated in Figure 5, catchbasins are installed by inserting a section of PVC pipe, with the sock, through the CSP tee catchbasin. Instead of using elbow pipes at the ends of the system, tee catchbasins were installed at end points and capped-off at one end. The use of capped-off tee catchbasins allow for future extension of the perforated pipe system.

The riser pipe segment of the catchbasins were capped and buried during backfilling with the native soil. After surface grading, the tops of the catchbasins were re-excavated and CSP extensions were coupled to the tee riser pipe. Overlapping clamps with bolted locking mechanism were used to join the vertical sections together. Following final landscaping, the protruding pipes sections were cut at the surface and 348 mm diameter cast iron covers were placed on top.

The Ontario Provincial Standard Drawing, 1200 mm diameter precast manhole (OPSD - 701.01M) were used at pipe intersections and corners. As illustrated in Figure 6, the manhole covers provide access for maintenance and allow runoff inflow into the pipe system. Protective covers were placed temporarily over the catchbasins and manhole covers until cleanup was completed.

4.2.4 Landscaping

Initial landscaping and swale grading was carried out using a small bulldozer. Later, a 50 to 100 mm thick layer of topsoil was spread over the area by hand and nursery sod was laid on top. Final grading provided shallow side and longitudinal slopes for conveying the overland runoff to the catchbasins. Granular fill and HL-3 asphalt was placed at driveway entrances.

4.2.5 Cleanup

The remaining native soil was removed from the site by dump truck. Road and driveway surfaces were cleaned using a street sweeper and a rotary brush sweeper. The remaining construction material was removed from the site and catchbasins and manholes were placed in operation by removing the protective covers.

4.3 Special Conditions Encountered

Several special conditions were encountered during the installation of the grass swale-perforated pipe system. For example, obstructions by a watermain required minor modifications to the pipe elevation. Backfilling over the tee catchbasin prior to final installation often introduced small amounts of debris (sediments, branches, etc.) into the CSP riser pipes. Some debris were also introduced during re-excavation and extension of the riser pipes segments and during the installation of the manholes.

Another special condition encountered was related to the placement of backfill over the top of the gravel trench and filter cloth. As discussed later, the native surficial soils excavated from the top of the trench consisted of loosely compacted fine-to-medium grain sand while the bottom consisted of silty blue-grey clay. Due to the nature of the excavation process, the surficial sandy material was deposited at the bottom of the pile while the silty clay material ended at the top. The silty clay was used as backfill over the top of the gravel trench and filter cloth. This material effectively creates a 'cap' between the swale and the trench reducing the infiltration capacity of the swale. Such conditions may in some cases create unwanted water ponding by preventing the infiltration of water.

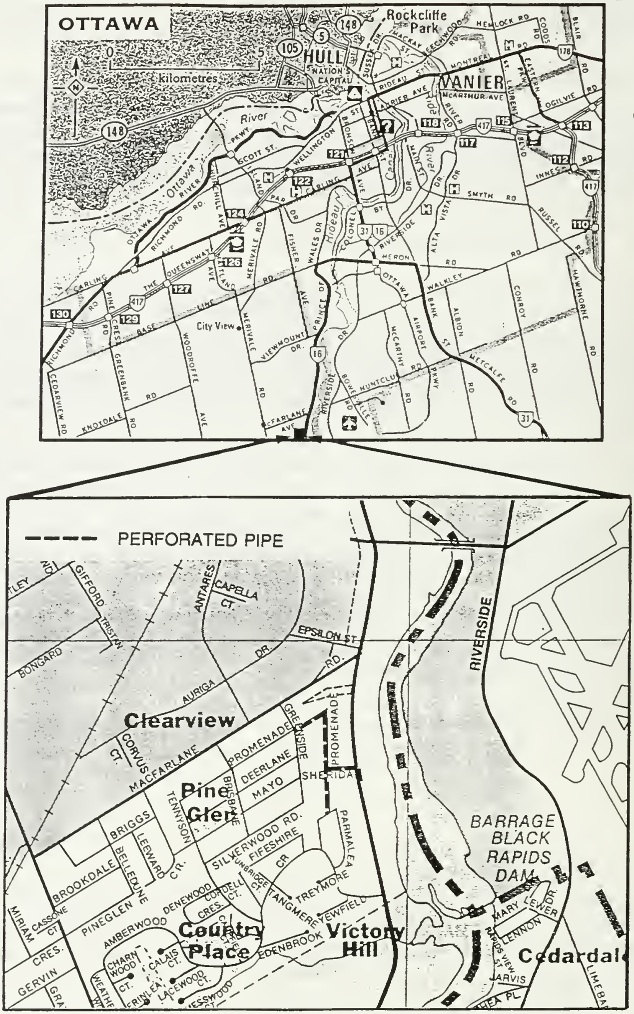

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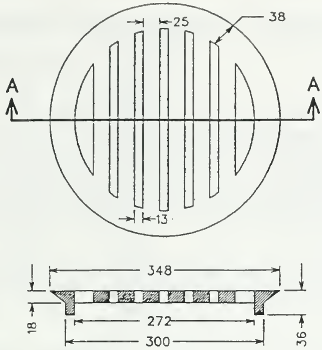
FIGURE 3. LOCATION OF CLEARVIEW DRAINAGE IMPROVEMENT SITE



SECTION A-A
CAST IRON COVER

PIPE SIZE TABLE	
P.V.C. PIPE	C.S.P.
250mm	400mm
300mm	400mm
375mm	450mm
400mm	450mm
450mm	600mm
500mm	600mm
525mm	675mm
600mm	675mm

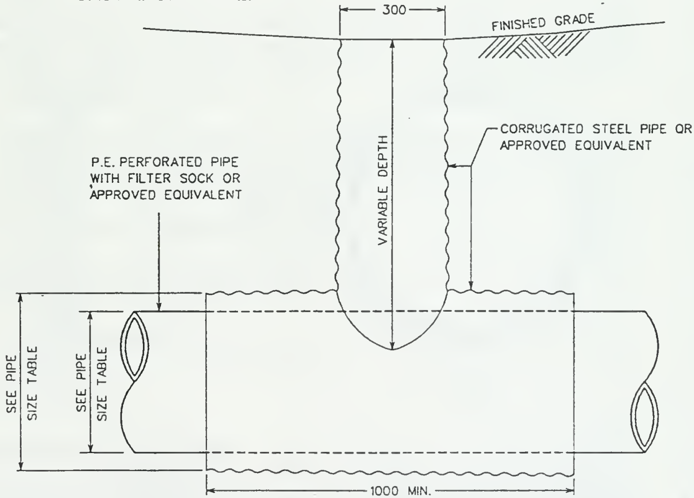


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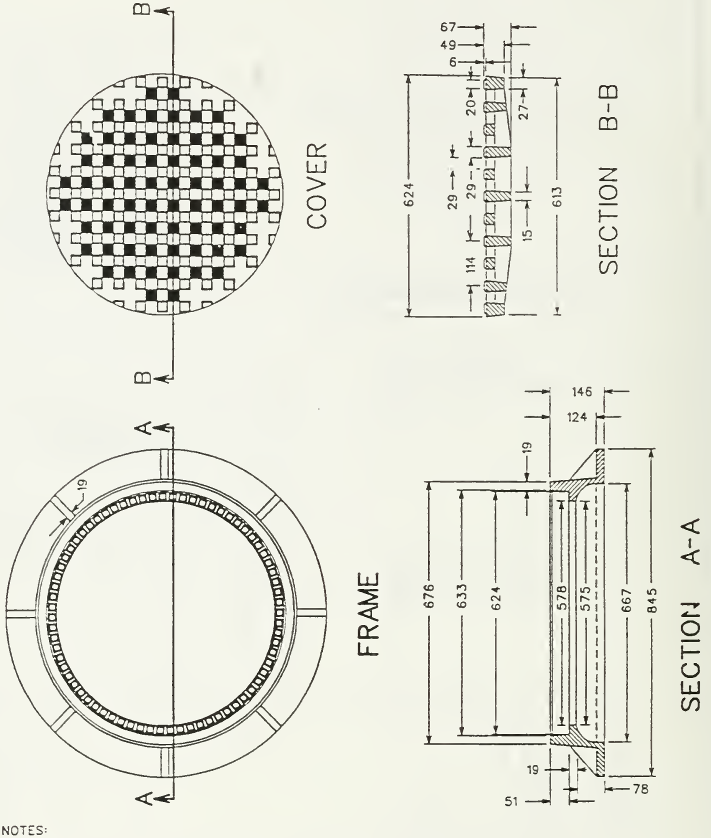
FIGURE 5. C.S.P. "T" TYPE CATCH BASIN

SOURCE: CITY OF NEPEAN



NOTES:

1. ALLOWABLE TOLERANCE FOR DIMENSIONS OF 300mm OR LESS IS ±3mm
2. ALLOWABLE TOLERANCE FOR DIMENSIONS GREATER THAN 300mm AND UP TO 900mm INCLUSIVE IS ±6mm
3. THE INITIALS OR MARKS OF THE MANUFACTURER ARE TO BE DISTINCTLY CAST IN RAISED LETTERS ON COVER
4. ALL DIMENSIONS IN MILLIMETERS UNLESS OTHERWISE SHOWN

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FIGURE 6. STORM MANHOLE AND FRAME COVER

SOURCE: CITY OF NEPEAN

5.0 THEORY AND MODELLING APPROACH

5.1 Methodology

A model for the simulation of the performance of a typical swale - perforated pipe drainage system is developed. The methodology is based on a water balance between the different components of the system. Figure 7 shows the flow path of water in a perforated pipe drainage system and the associated design parameters. The system is analyzed on the basis of the characteristics of a single typical lot, pipe, and trench configuration.

A flow chart of the modelling methodology which was adopted, is shown in Figure 8. The different steps of the modelling process are sequentially described as follows:

Precipitation - __Runoff__

Surface runoff from a typical lot can be generated from various methods or models and is not the subject of this study. However, for all intents and purposes, a method of calculation, known as the coefficient method (similar to the Rational Method) is provided in the model developed herein. Alternatively, a runoff hydrograph obtained from other sources can also be used as an input.

To simulate a surface runoff hydrograph, the coefficient method used by the model requires a rainfall hyetograph (P) and lot characteristics such as area (A_l), initial abstraction (Ia) and a runoff coefficient (C_l) based on imperviousness.

The derivation of the runoff hydrograph is as follows:

After depleting the initial abstraction (Ia) from the first rainfall increments, the lot runoff at any time step 'i' can be obtained from:

$$R_i = P_i \, C_l \, A_l \qquad (1)$$

Runoff - __Inflow to Pipe Catchbasin__

As the resulting runoff from a lot is conveyed to the grass swale, some infiltration losses occur. Infiltration through the grass swale is estimated as the product of the soil permeability and the wetted area produced by the flow. This infiltration is then subtracted from the total incoming runoff to give the rate of flow entering the perforated pipe system via the catchbasin. Infiltration in the swale is divided into

two parts: (i) infiltration on the sides of the swale as surface runoff is collected and (ii) infiltration which occurs along the swale as runoff is carried to the catchbasin. The first part of infiltration is expressed as:

$$Q_{s1} = 2\ K_s\sqrt{1 + Z^2}\ D\ L \tag{2}$$

where K_s = swale's soil permeability, which represents the minimum or equilibrium soil infiltration capacity, D = maximum depth in the swale defined by the width and side slopes, L = longitudinal swale length, and Z = swale side slope (V:1, H:Z). This relationship assumes that runoff is collected equally on both sides of the swale.

The infiltration along the swale length is obtained by:

$$Q_{s2} = (y_1 + y_2)\ K_s\ L\sqrt{1 + Z^2} \tag{3}$$

where y_1 and y_2 are the respective upstream and downstream depths of flow in the swale based on Manning's equation.

Finally, the total infiltration rate in the swale during any time step "i" is given by:

$$Q_{s_i} = Q_{s1_i} + Q_{s2_i} \tag{4}$$

This total infiltration is then subtracted from the total incoming runoff to give the rate of flow entering the perforated pipe system via the catchbasin.

$$Q_{cb_i} = R_i - Q_{s_i} \tag{5}$$

Flow Captured by Swale - Inflow to Pipe Delayed by Infiltration

Some of the runoff volume infiltrated at the swale's surface can eventually reach the pipe by means of infiltration. This flow can then be combined with the flow which entered directly into the catchbasin, and the total can either be exfiltrated through the pipe's orifices and stored in the trench or be conveyed by the pipe to downstream segments.

Conservatively, if it is assumed that all the flow captured by the swale can make its way to the perforated pipe then this additional inflow 'Q_b' (from backfill) to the pipe is, in fact, equal to the flow infiltrated in the swale (Q_s) delayed by a time 'T_{Qb}' determined by the thickness 'D_b' of backfill material above the pipe and its hydraulic conductivity 'K_b'. Thus,

$$Q_b = Q_s \quad \textit{delayed by} \quad T_{Q_b} = \frac{D_b}{K_b} \tag{6}$$

In order to minimize this additional flow component, the backfill material and thickness between the swale's surface and the pipe should be such as to not allow water to reach the pipe within the design storm duration. As such:

$$\frac{D_b}{K_b} > \textit{Design Storm Duration} \tag{7}$$

Total Inflow to Pipe

The total inflow to the perforated pipe is the sum of the flow directly captured by the catchbasin and the flow captured by the grass swale delayed by the infiltration process. Thus,

$$\begin{aligned} Q_p &= Q_{cb} && \textit{for } t < T_{Qb} \\ Q_p &= Q_{cb} + Q_b && \textit{for } t \geq T_{Qb} \end{aligned} \tag{8}$$

Total Inflow in Pipe - Outflow through Pipe Perforations

The pipe, being perforated, allows exfiltration of water along its length for storage in the gravel bed underneath. The flow rate exfiltrated through the perforations is computed based on the depth of flow in the pipe and is a function of pipe length and slope, size and shape of the orifices, number of orifices and their orientation around the circumference, number of orifices per unit pipe length.

The theoretical orifice equation is initially used to relate discharge (q) through an orifice area (a) to the water depth (y) above it.

$$q = ca\sqrt{2gy} \tag{9}$$

Where g = acceleration due to gravity, and c = discharge coefficient, that mainly accounts for energy losses and contraction of the flow area. Discharges through all orifices along the pipe are computed using the same equation and then summed to yield the total discharge exfiltrated out of the perforated pipe.

In order to compute discharges through the orifices (which depends on water depth) it is necessary to simulate the water surface profile within the pipe. This latter

depends on discharge in the pipe, pipe length, pipe slope and on the change of the flow rate along the pipe to account for water losses through pipe perforations. The water surface profile within the pipe can be simulated by assuming either uniform or gradually varied flow. If uniform flow is assumed, an iterative process is adopted to compute flow depth in the pipe based on Manning's equation:

$$Q = \frac{A}{n} R^{2/3} S^{1/2} \tag{10}$$

where Q = flow rate in the pipe at a given distance from the catchbasin,
A = cross-sectional area of flow in the pipe,
n = Manning's roughness coefficient,
R = hydraulic radius, and
S = pipe slope.

If non-uniform flow is assumed, gradually - varied flow computations are performed as follows:

First, critical and uniform depths have to be computed and compared. If the uniform depth is smaller than the critical depth the pipe is on a steep slope and computations start from the upstream end at critical depth and proceed downstream. Otherwise, the pipe has a mild slope and computations start downstream and proceed upstream. In this instance, since the outflow from the pipe at its downstream end is not known, an iterative procedure is adopted. An outflow rate is assumed and the inflow rate at the upstream end of the pipe is computed. The outflow rate at the downstream end is then adjusted until the computed inflow rate matches the actual given value.

Discharges through all orifices along the pipe are computed using the corresponding flow depths and equation (9). The total flow rate exfiltrated out of the pipe is then calculated as:

$$Q_c = \sum_{i=1}^{p} q_i \tag{11}$$

where p = the number of pipe perforations under water. In the case where the trench volume and its surrounding soil cannot handle the incoming flow Q_c, the flow out of the pipe perforations is made equal to the infiltration rate capacity of the soils around the trench. Under these circumstances, the captured flow rate is controlled by the trench rather than by the pipe's orifices. The difference between the discharge in the pipe and the flow rate captured by the orifices and/or the trench is conveyed to the next downstream pipe segment and can be considered as an overflow 'Q_{povf}'.

Flow Captured in Trench - Exfiltration to Native Soil

Water stored in the trench is exfiltrated to the surrounding soil, which is eventually infiltrated to the groundwater table. The available storage area in the trench is assumed to be only the volume below the pipe invert. The total trench volume (V_t) is calculated as the product of the bedding depth D_t, the trench width (W_t), length of the trench (L_t), and the void ratio (e) of the trench's granular material.

$$V_t = D_t\ W_t\ L_t\ e \tag{12}$$

The exfiltration rate of water passing from the trench to the native soil (Q_e) is calculated using Darcy's Law:

$$Q_e = r\ K_{ns}\ (W_t + 2D_t)\ L_t\ i \tag{13}$$

where K_{ns} = native soil hydraulic conductivity,
i = hydraulic gradient, assumed to be 1.0 m/m, and
r = a reduction factor to account for clogging.

Values for the reduction factor to account for clogging vary with the years of use. Typical values, taken from the literature are shown in Table 3.

Table 3: Years of Use and Infiltration Capacity Reduction Ratio

Years of Use	Less Than 5 Years	10 Years	30 Years	50 Years
Reduction Factor	0.9	0.8	0.5	0.3

Flow Captured in Trench - Trench Overflow

As indicated above, the pipe flow which does not enter the trench through the pipe's orifices is conveyed to the next downstream pipe segment as 'Q_{povf}'. In addition, it is also known that water in the granular material of the trench below the pipe can also travel along its length and thus be conveyed to the next downstream trench segment. This trench outflow (Q_{tovf}) can be computed using Darcy's Law:

$$Q_{tovf} = K_g \, (W_t \, Y_t) \, i \tag{14}$$

where K_g = hydraulic conductivity of the trench granular material,
W_t = bottom width of trench,
Y_t = flow depth in the trench, and
i = hydraulic gradient.

Total Flow to Downstream Pipe/Trench Segment

Ultimately, the volume of water which is not stored in the trench below the pipe or infiltrated in the native soil will be conveyed to the next downstream pipe and trench segment. As shown above this flow is made up of two components 'Q_{povf}' and 'Q_{tovf}'. Therefore, the total overflow 'QT_{ovf}' can be defined by;

$$QT_{ovf} = Q_{povf} + Q_{tovf} \tag{15}$$

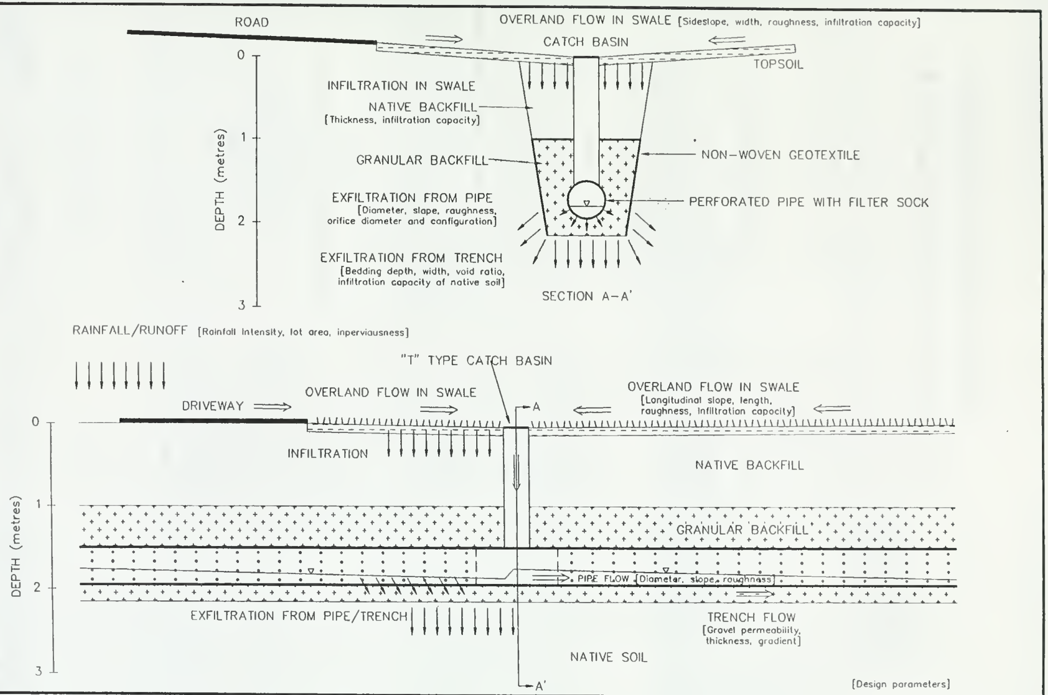

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FIGURE 7. SCHEMATIC OF GRASS SWALE PERFORATED PIPE DRAINAGE SYSTEM

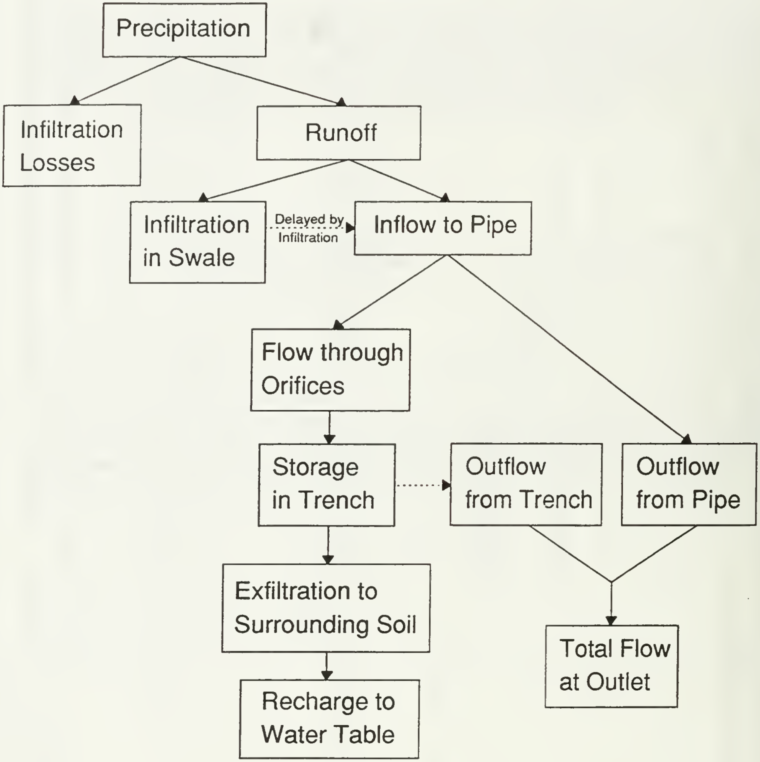

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FIGURE 8. FLOW CHART FOR SYSTEM MODELLING METHODOLOGY

6.0 LABORATORY AND FIELD TESTS

6.1 Perforated Pipes Experimental Program

Laboratory tests were conducted to investigate the hydraulic performance of perforated pipe systems under different design scenarios. Hydraulic performance is evaluated in terms of outflow through pipe perforations for various pipe characteristics and inflow rates.

6.1.1 Experimental Set-up

Experiments were conducted in the Hydraulics Laboratory of the University of Waterloo and the experimental set-up is shown in Figure 9 and Photos in Appendix D. A smooth-wall perforated pipe section 3.66 m (12 ft) long was placed inside a 1 m confining wood box and surrounded by gravel.

The pipe was supported at both ends and at the middle. The pipe joints at the ends were sealed along the circumference with Silicone glue to prevent leakage. A metal grate was installed underneath the box to collect the flow exfiltrated from the pipe through the perforations and through the gravel bed. Two reservoirs, simulating manhole structures, were installed at the upstream and downstream ends of the pipe. Manometers were installed inside both reservoirs to measure the water levels. Pre-calibrated triangular sharp-crested weirs were used to measure inflow to the pipe and the flow through the metal grate (i.e. outflow through pipe perforations). Head-discharge curves for both weirs are shown in Figure 10. Outflow at the downstream end of the pipe was computed based on mass balance considerations.

6.1.2 Experimental Procedure

The pump was initially started to circulate water until the upstream and downstream reservoirs were filled. Water in the trench was then allowed to drain and zero manometer readings were recorded. The pump was then restarted and water circulated until a steady state condition was reached. At this stage water depths at the upstream and downstream reservoirs as well as the upstream and downstream sharp-crested weirs were taken. Measurements were taken for several inflow rates ranging between 2 and 12 l/s.

6.1.3 Variations Tested

The objective of the experimental program was to test the performance of the perforated pipe in terms of exfiltration rates under different design scenarios. The

different variations tested are listed in Table 4. Variations included different pipe sizes, different orifice sizes and different pipe slopes. Two pipe sizes (300 and 450 mm), which are typical sizes for perforated pipe applications, were tested. Pipe slope varied between 0 and 2%, which cover a wide range of slopes that would occur in storm sewer designs.

Table 4: Laboratory Test Conditions

TEST VARIATIONS						
PIPE DIAMETER PERFORATIONS FILTER CLOTH GRAVEL		300 mm 5/16" no yes	300 mm 5/16" yes no	300 mm 5/16" yes yes	300 mm 1/2" yes yes	450 mm slots yes yes
PIPE SLOPES	0 %	0,50,100 mm baffles	no	0,50,100 mm baffles	0,50,100 mm baffles	0,50,100 mm baffles
	1 %	0,50,100 mm baffles	no	0,50,100 mm baffles	0,50,100 mm baffles	0,50,100 mm baffles
	2 %	0,50,100 mm baffles	0,50,100 mm baffles	0,50,100 mm baffles	0,50,100 mm baffles	0,50,100 mm baffles

Initial tests were conducted without cloth or gravel to determine whether these latter have any effect on the exfiltration rate through the perforations. Tests were also performed with a baffle at the downstream end of the pipe to evaluate changes in exfiltration rates through the pipe perforations by artificially raising water levels. The effects of baffles with heights of 1/6 and 1/3 of pipe diameter were tested for the different pipe slopes and different pipe sizes.

6.1.4 Results and Discussion of Laboratory Tests

The complete set of results for all conditions tested are included in Appendix D. Exfiltration rates are plotted against inflow rates for different pipe slopes in Figure 11. The figure shows that flow rates out of pipe perforations increase with a decrease in pipe slope. This is explained by the fact that exfiltration rates through pipe perforations are mainly a function of the depth of flow above the orifice (lower slopes, higher flow depths). On the other hand, although the total captured flow increased with higher inflow rates, it is seen that the percent captured decreases.

The installation of baffles or weirs at the downstream end of the pipe, which results in artificially increasing flow depth upstream in the pipe, increases the flow rate through the pipe's perforations. Figure 12 shows exfiltration rates against inflow rates for the conditions with and without baffles. The figure clearly shows that baffles provide significant improvements in terms of increasing exfiltration rates out of the pipe, particularly for low inflow rates.

The effect of orifice size on exfiltration rates was also examined. Figure 13 shows that exfiltration rates increase with an increase in the orifice cross-sectional area. However, the results obtained indicated that the increase in discharge is not proportional to an increase in orifice size. In the experimental program, the orifice area was increased by a factor of approximately 2.6 when the orifice size changed from 7.9 to 12.7 mm. However, exfiltration rates increased by a factor of only 1.5 to 1.8. These results suggest that discharge coefficient vary with discharge or flow depth in the pipe and orifice size.

6.2 Infiltration Field Tests

6.2.1 Swale Infiltration Field Tests

Five in situ field infiltrometer tests were performed at the Promenade Avenue site to determine representative infiltration rates for the grass swales. A single ring cylinder infiltrometer, made from a 20 cm diameter PVC pipe, was used. The edge of the cylinder was bevelled to reduce soil disturbance when driven into the soil. A schematic of the apparatus used is shown in Figure 14.

The infiltrometer was installed by laying a piece of wood on the infiltrometer and pounding it in with a hammer. Depths of penetration varied between 8 and 12 cm, depending upon thickness of the top soil cover. At each test site, attempts were made to ensure that the infiltrometer penetrated the entire thickness of the more permeable topsoil layer to prevent errors that can be induced because of lateral leakage.

Infiltration rates were determined using the constant head method. Constant water depths in the infiltrometer were maintained using a Mariotte siphon (Figure 14). The principle of the Mariotte siphon is that the pressure inside the bottle at the level of the bubble tube is at atmospheric value, which then maintains the water surface in the infiltrometer at the same height as the end of the bubble tube. Water depths at the various sites varied between 2.5 and 5.0 cm, depending upon the length of the grass and unevenness of the surface.

Infiltration rates were calculated from the rate of fall of the water level in the Mariotte siphon reservoir with readings being taken every two minutes. Constant

rates of infiltration were achieved after 14 and 20 minutes of testing. Limiting curves defining the maximum possible rate of infiltration versus time are shown in Figure 15.

6.2.2 Trench Infiltration Field Test

The main purpose of this test was mainly to establish the infiltration capacity of the native soil under the trench. The test was conducted along Brisbane Street between Silverwood and McFarlane Roads in Nepean. Water was injected in a catchbasin at the upstream end of the system using a fire hydrant and a hose. The flow rate was regulated and measured using a bedger volumetric flow meter. Discharge at the downstream end of the system was measured with a 90° triangular weir. Depths, which were measured using a stilling well float assembly, were translated into flow rates using a pre-developed calibration curve. The following data, considered to be representative of the physical system, were used in the interpretation of the results.

Perforated Pipe Data:

- Pipe length = 338 m
- Pipe diameter = 0.3 m
- Orifice size = 7.9 mm (5/16 in)
- Orifice spacing = 5.0 cm

Trench Data:

- Trench width = 0.75 m
- Trench bedding thickness = 0.15 m
- Void ratio of the granular material = 40%

For the test, an inflow rate of 9.8 l/s was held constant for approximately 1.5 hours over which the flow rate at the outlet stabilized at approximately 6.3 l/s. Figure 16 shows the measured inflow and outflow hydrographs.

Assuming that the difference between the measured inflow and outflow rates are attributed to the water infiltrating into the native soil beneath the trench, then the infiltration capacity of this soil can be estimated at 3.5 cm/hr which is consistent with the types of soil (sand/silty till) which were identified at the site (Section 9).

From the measurements shown in Figure 16 it can also be derived that the total volume injected in the system was approximately 53 m^3 while the volume measured at the outlet was only 27 m^3. This represents a 50% loss of the injected water.

6.2.3 Results and Discussion of Field Tests

The results of the swale infiltration tests (Figure 15) indicate that initial infiltration rates over a grassed surface (swale) can vary from 3 cm/hr to 13 cm/hr. However, it was also found that constant infiltration rates from 1 cm/hr to 3 cm/hr were achieved after approximately 15 to 20 minutes of testing. For modelling purposes, a conservative infiltration rate of 1 cm/hr can be used for the grass swale.

The results of the trench infiltration tests (Figure 16) indicated that design infiltration rates of native soils beneath the pipe trench can be based on literature values. As such, for the site tested, sand/silty till soils were found and an infiltration rate of 3.5 cm/hr was measured.

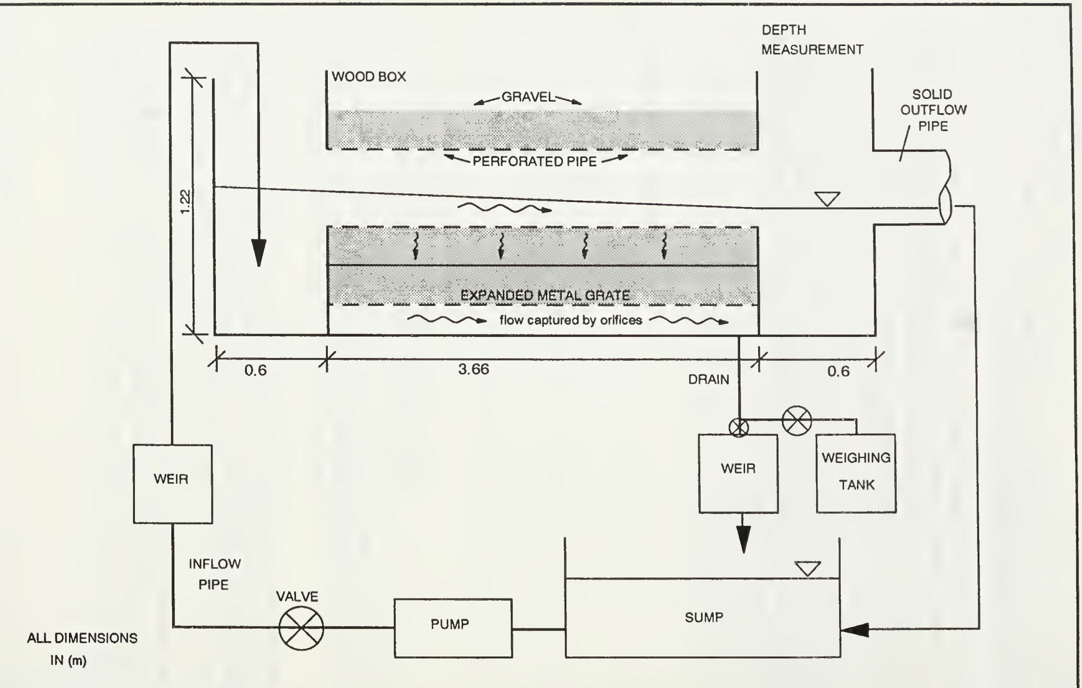

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FIGURE 9. SCHEMATIC OF HYDRAULIC LABORATORY SET-UP

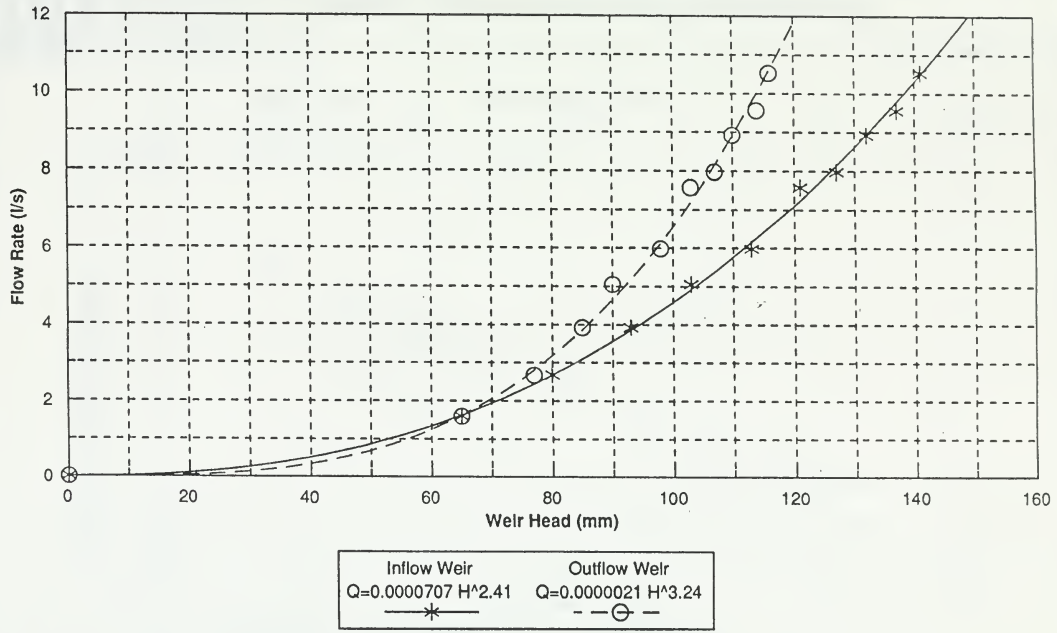

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FIGURE 10. CALIBRATION CURVES OF V-NOTCH WEIRS USED TO MEASURE THE FLOWS IN THE LABORATORY SET-UP

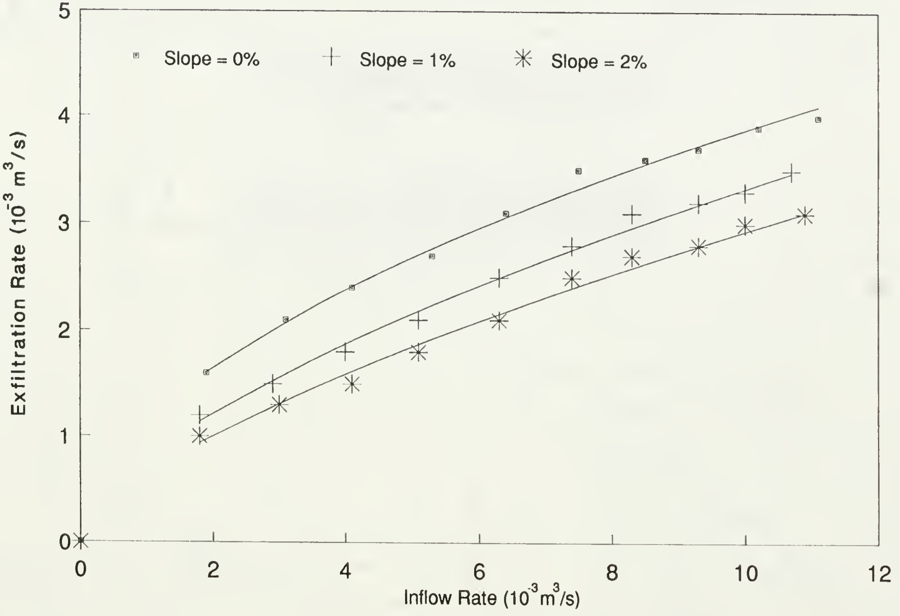

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FIGURE 11. EFFECT OF PIPE SLOPE ON EXFILTRATION RATES

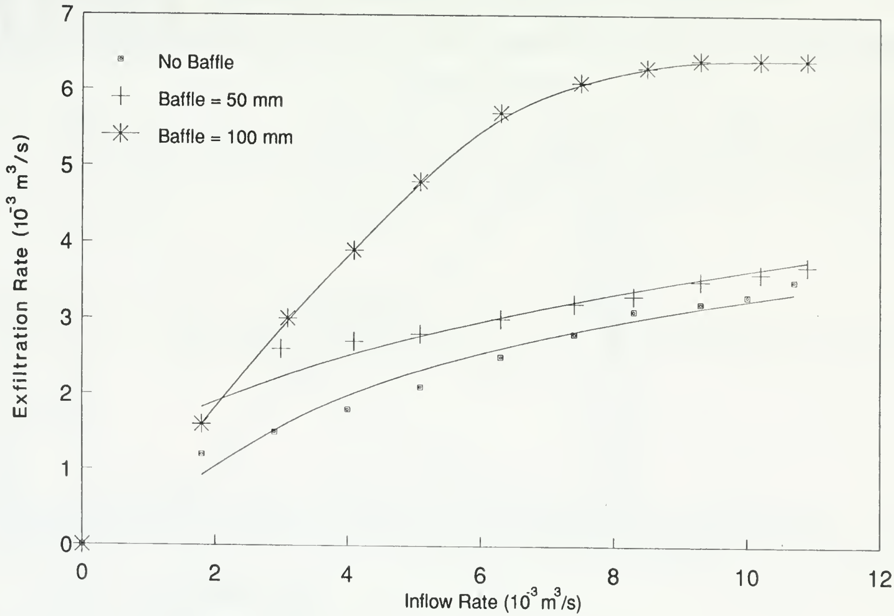

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FIGURE 12. EFFECT OF INSTALLING A BAFFLE ON EXFILTRATION RATES

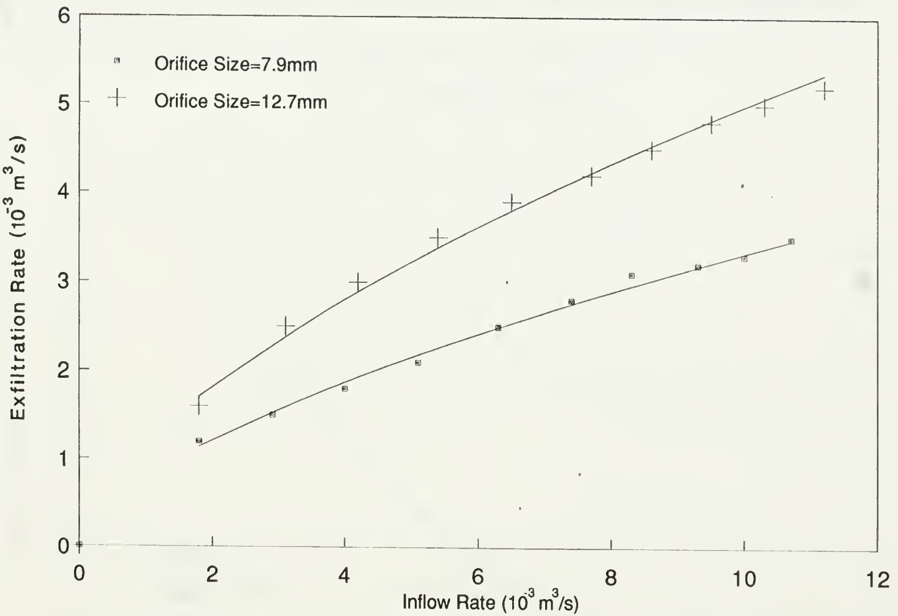

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FIGURE 13. EFFECT OF ORIFICE SIZE ON EXFILTRATION RATES

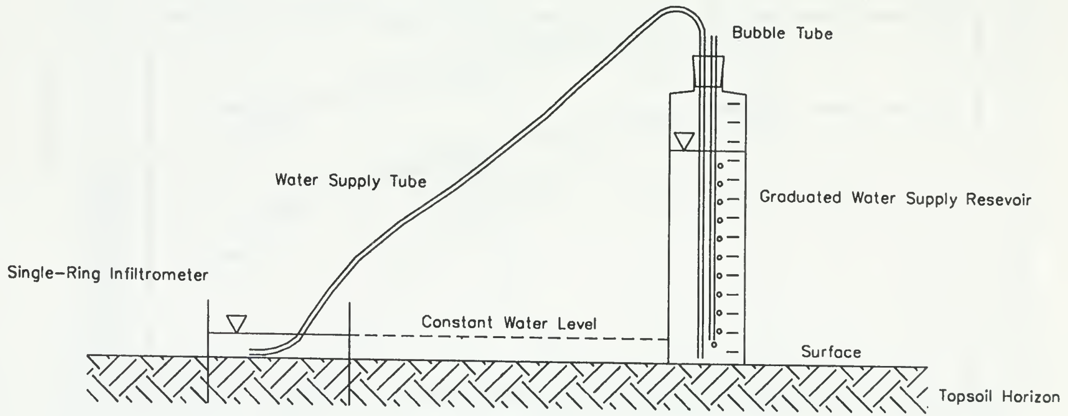

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FIGURE 14. SCHEMATIC OF SINGLE RING INFILTROMETER AND MARIOTTE SIPHON APPARATUS

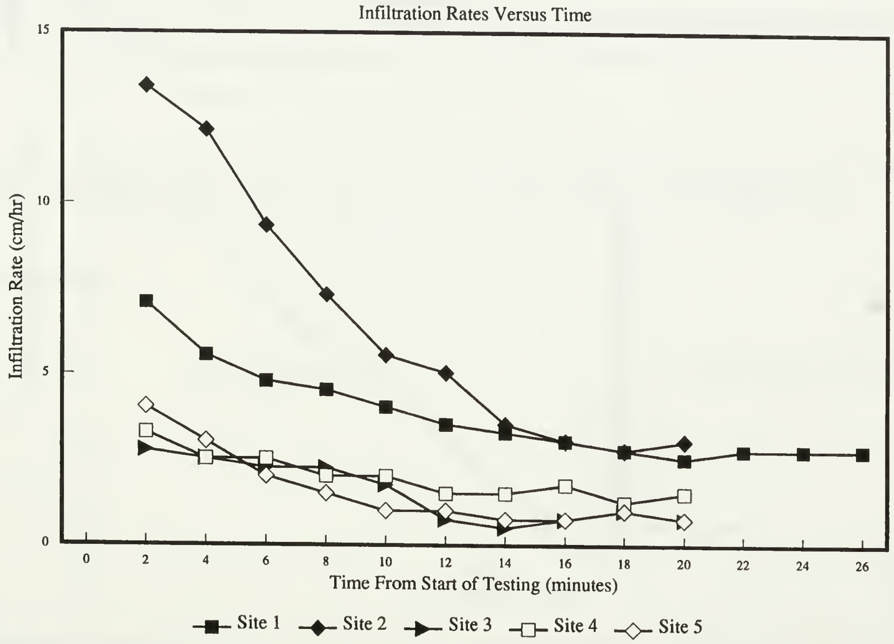

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FIGURE 15. INFILTRATION CAPACITY CURVES - PROMENADE AVENUE

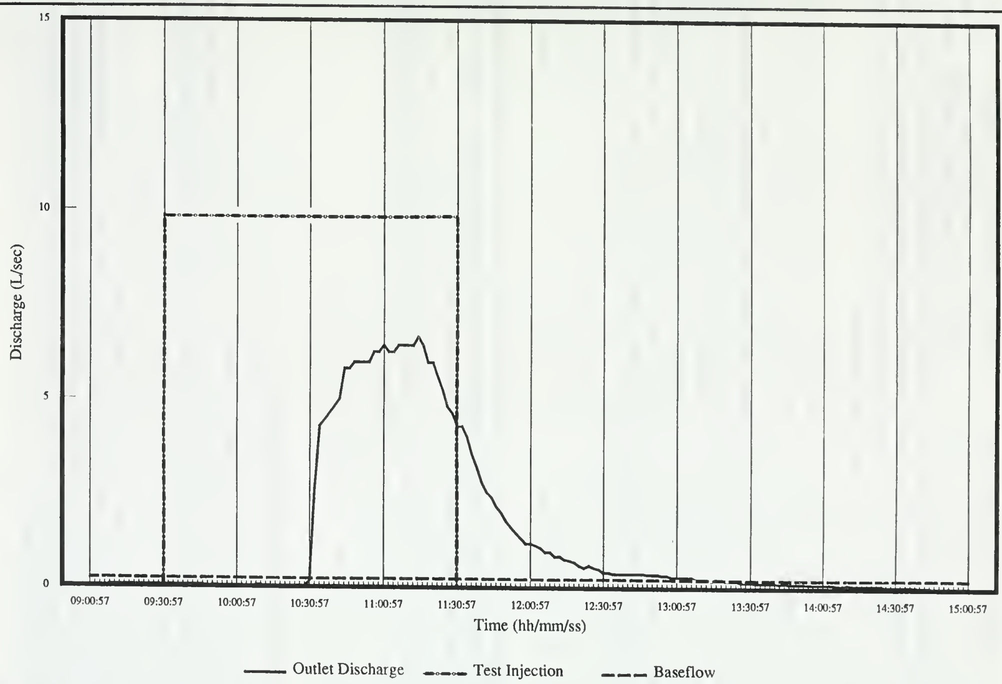

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FIGURE 16. TRENCH INFILTRATION TEST

7.0 MODEL CALIBRATION AND VALIDATION

In Section 5, a theoretical model for the ANalysis of grass SWales And Perforated Pipe Systems (ANSWAPPS) was derived. To better understand these processes, a series of laboratory and field tests were conducted and are described in Section 6. From these findings and other field data collected during a three month monitoring campaign the model is calibrated and validated.

7.1 Calibration

The interrelation between the various hydrologic/hydraulic components involved in the operation and performance of grass swale-perforated pipes can be very complex and, as such, the calibration of the individual processes can be difficult when taken separately. Nevertheless, based on the results of the field investigations the infiltration rates of the swale and the soils surrounding the trench were estimated. The hydraulic laboratory investigations were mainly used to calibrate the discharge coefficient of the pipe's orifices.

7.1.1 Swale Infiltration

As described in Section 5, the infiltration at the level of the grass swale is a function of the swale's physical characteristics (e.g. length, slopes, depth) and its soil's infiltration capacity, hence the only parameter to be calibrated.

From field tests it was found that equilibrium infiltration capacity 'K_s' for the grass swale can vary from location to location and may have a value of 1 cm/hr to 3 cm/hr.

Field data for this test is provided in Appendix E.

7.1.2 Perforated Pipe Hydraulics

This portion of the model calibration is certainly the most extensive as the flow along a perforated pipe can be, hydraulically, very complex. In order to simulate the behaviour of water flowing along a perforated pipe, the modelling approach must consider many factors such as pipe diameter, pipe slope, pipe roughness, orifice characteristics (size and arrangement), inflow rates, flow depths, etc.

The orifice equation was used to relate discharge out of the pipe perforation to flow depth above the orifice. The water surface profile in the pipe was calculated from Manning's equation, accounting for the decrease in pipe discharge downstream of each ring of perforations.

Initially, a discharge coefficient of 0.63, which is typical for circular orifices, was used and the simulated exfiltration rates were compared to experimental values. These results are shown in Figures 17 to 20 for two different orifice sizes and pipe slopes. According to these figures, compared to measured values, simulated exfiltration rates were shown to be consistently overestimated. This would suggest that the actual value for the coefficient of discharge is less than the value used of 0.63.

The discharge coefficient was then varied over a wide range and the summation of the squares of relative errors between simulated and observed exfiltration rates was minimized. This exercise yielded a discharge coefficient value of 0.5 and 0.34 for the 7.9 and 12.7 mm orifice sizes respectively. These results support the conclusion drawn from the experimental program stating that different orifice sizes have different discharge coefficients.

Figures 17 to 20 show that with different discharge coefficients for each orifice sizes the agreement between observed and simulated exfiltration rates is improved. However, it was felt that these results could not be generalised and were somewhat in contradiction with findings from others. Based on this, it was concluded that the discharge coefficient may in fact have an maximum value of C_{max}=0.63 for a given depth of flow but may be less for smaller depths. This is also suggested in some literature.

To investigate this assumption different interpolating functions (elliptic and sine), were used to vary the orifice discharge coefficient from a value of 0.00 to 0.63 between flow depths of 0.00 to H_{max} which is to be obtained through calibration and may somewhat depend on the diameter of the orifice. These functions are expressed mathematically as follows;

$$C = C_{max} \sqrt{1 - (1 - \frac{H}{H_{max}})^2} \qquad \textbf{(16)}$$

$$C = C_{max} \, Sin \frac{\pi}{2} \, \frac{H}{H_{max}} \qquad \textbf{(17)}$$

and are shown graphically on Figures 21 and 22 for the 7.9 and 12.7 mm diameter orifices, respectively.

Using the hydraulically related component of the model, flow rates exfiltrated out of the pipe were computed using the elliptic and sine discharge coefficient interpolating functions. These are plotted against inflow rates for four different test conditions (orifice size = 7.9 and 12.7 mm, pipe slope = 1 and 2%) in Figures 23 to 26. Results obtained using the elliptic interpolating function are marginally better than those

obtained using the sine function. This conclusion is confirmed by comparing the summation of the squares of relative errors induced using both methods, shown in Table 5.

Table 5: Summation of the Squares of Relative Errors Induced Using Elliptic and Sine Interpolating Functions

Interpolating Function	Orifice Diameter d = 7.9 mm		Orifice Diameter d = 12.7 mm		Average
	Pipe Slope S = 1%	Pipe Slope S = 2%	Pipe Slope S = 1%	Pipe Slope S = 2%	
Elliptic	0.070	0.086	0.099	0.044	0.075
Sine	0.037	0.116	0.159	0.098	0.103

Through an optimization technique to minimize the error between simulated and measured exfiltration rates, discharge coefficients were found to vary over the depths shown in Table 6. The table shows different values for these depths not only for the two interpolating functions considered but also for the two orifice sizes. In fact, orifice discharge coefficients are expected to vary with orifice size as reported by Medaugh and Johnson (1940) among others.

Table 6: Flow Depths (in Millimetres) Over Which the Orifice Discharge Coefficient Was Varied
(Obtained Through Optimization)

Interpolating Function	Orifice Diameter 7.9 mm	Orifice Diameter 12.7 mm
Elliptic	0 - 55	0 - 180
Sine	0 - 43	0 - 83

Although both interpolating functions perform well, the elliptical function gave slightly less error and is recommended for use. For orifice sizes which were not tested, it is recommended that the depths over which the orifice coefficient should vary should be interpolated based on the results giving in Table 6. Extrapolating for larger orifice sizes should be done with care.

Exfiltration rates computed based on a constant discharge coefficient of 0.63 as well as those obtained using a variable discharge coefficient (elliptic function) are plotted against observed data in Figure 27. The figure clearly shows a much better agreement between the variable discharge coefficient results and observed exfiltration rates.

In the model, Manning's equation was used to simulate the water surface profile along the pipe from orifice to orifice. However, this assumes a uniform flow condition and the use of a short pipe in the laboratory (less than 4.0 m) with the presence of reservoirs at the upstream and downstream ends are factors that might affect the validity of this assumption. To verify this, the water surface profile was generated using gradually-varied flow computations and results are compared to those obtained using Manning's equation. Simulated water surface profiles using both approaches were found to be similar (refer to Figure 28).

According to Figure 28, compared to the one generated using Manning's equation, the water surface profile computed using gradually varied flow theory started at a higher depth but also had a higher slope, implying a decrease in flow depth at a faster rate. These two factors counterbalanced each other resulting in reasonably similar water surface profiles, and therefore, similar exfiltration rates out of the pipe (see Figure 29). It is also expected that these differences would be even smaller for longer pipes such as the ones in the field.

7.1.3 Trench Exfiltration

As described in Section 5, the exfiltration process between the water stored in the trench (below the pipe invert) and the native soil depends mainly on the trench's physical characteristics (e.g. length, width, depth and void ratio) which are known and on the native soil's infiltration capacity, hence the only parameter to be calibrated.

From field tests it was found that the infiltration capacity of native soils (not affected by groundwater) can be based on textbook values. Typical soil infiltration rate values are given in Table 7.

Table 7: Soil Infiltration Rates

Soil Texture	SCS Soil Group	Minimum Infiltration Rate (cm/hr)
Sand	A	21.0
Loamy Sand	A	6.1
Sandy Loam	B	2.6
Loam	B	1.3
Silty Loam	C	0.7

7.2 Model Validation

As indicated above, except for the pipe hydraulics, it is difficult to separate and calibrate the individual hydrological processes involved in the operation of grass swale-perforated pipe systems. Consequently, the validation of the modelling procedure is done on the premise that all processes are interdependent and are combined to create a type of "black box" model.

It is important to remember at this point that the modelling approach described in Section 5 was derived to simulate the hydrologic/hydraulic processes occurring within a limited section of a grass-swale perforated pipe system. That is, the model does the analysis for one typical lot with one catchbasin and typical swale, pipe and trench characteristics. Because of this, it is difficult to validate the model based on a comparison of simulated peak flows (for one lot) with measured peak flows (for an entire subdivision). However, an indicative comparison can be made if the effective runoff coefficients 'C_{eff}' obtained from the model (ANSWAPPS) are used in a more conventional hydrologic model such as OTTHYMO-89. This will be demonstrated below.

With the rainfall data which was collected at two sites in the City of Nepean for a period of three months (see Section 9), the model's capacity to compute overall runoff coefficients was tested. The results were compared with the ones obtained from flow measurements. The two sites (referred to as Heart's Desire and McFarlane) at which data were collected are further described in Section 9.

7.2.1 Variable Orifice Discharge Coefficient

To validate the modelling approach of a varying orifice coefficient, simulations were performed using the experimental data obtained for the slot perforations. In this case, the pipe diameter was 0.450 m and the perforations were 3.5 cm long and 0.5 cm wide. The elliptic discharge coefficient interpolating function with a maximum depth of 0.18 m was used. Simulated and observed exfiltration rates are plotted against inflow rates for a pipe slope of 1 and 2% in Figures 30 and 31 respectively. Both figures show reasonably good agreement between observed and simulated exfiltration rates.

7.2.2 Effective Runoff Coefficient

Simulations with ANSWAPPS were performed for the two monitored sites and with their respective typical lot, swale and pipe characteristics. Rainfall data which was also measured at both sites was used to generate the surface runoff.

The physical characteristics of both systems were comparable, except for the native soil which was silty loam ($f_c \simeq 1.0$ cm/hr) in Heart's Desire and silty sand ($f_c \simeq 3.0$ cm/hr) in McFarlane. Other data, used in the simulations, are given as follows:

- initial abstraction = 2.0 mm
- imperviousness draining directly to swale = 15% and 17% for Heart's Desire and McFarlane respectively
- swale side slope = V:1, H:30
- swale top width = 6.0 m
- swale Manning's roughness coefficient = 0.3
- pipe diameter = 0.3 m
- pipe Manning's roughness coefficient = 0.012
- swale/pipe longitudinal slope = 1%
- diameter of pipe perforations = 7.9 mm
- spacing between rings of perforations = 5.0 cm
- trench width = 0.75 m
- trench depth = 0.175
- void ratio of the granular material in the trench = 0.40

For Heart's Desire Subdivision the drainage area was 13.6 ha, the total length of pipes was 3045 m, and the total number of lots was 70. This gave an average lot size of 0.195 ha and a pipe length of 43.5 m per catchbasin (or CB per lot).

For the McFarlane-Pine Glen Subdivision the drainage area was 10.02 ha, the total length of pipes was 2685 m and the total number of lots was 39. This resulted in an average lot size of 0.257 ha and a pipe length of 68.8 m per catchbasin for this

particular subdivision. The above data were used to perform the single lot analyses with ANSWAPPS.

Based on the simulated overflow volume carried to downstream pipe segments, an effective average volumetric runoff coefficient was computed. It is important to distinguish between these runoff coefficients, computed based on overflow and the lot runoff coefficient, which is a function of lot imperviousness. The former is a function of the lot runoff coefficient and also the characteristics of the grass swale-perforated system.

For some selected events, Tables 8 and 9 show observed and simulated runoff volumes, runoff coefficients for Heart's Desire and McFarlane-Pine Glen Subdivisions respectively. Unlike the results obtained for Heart's Desire, which are generally in good agreement with observed data, observed runoff coefficients in McFarlane-Pine Glen Subdivision are significantly higher than computed values. This is mainly explained by the fact that the system in McFarlane was partly affected by a significant baseflow, which resulted in higher volumetric runoff coefficients (this aspect is further discussed in Section 9).

7.2.3 Peak Flows

The computed overflow coefficients, which were obtained based on a typical single lot analysis, were used to simulate overflow hydrographs for the whole watershed using the OTTHYMO-89 hydrological model. The NASHYD sub-model with its proportional loss coefficient option was used to conduct the exercise.

It is very important to point out that the final runoff coefficients given by the ANSWAPPS model are based on a comparison of total runoff versus total rainfall obtained from a given event. In fact, based on the conditions of the grass swale, water in the trench and infiltration in surrounding soils, ANSWAPPS calculates a new runoff coefficient for each simulation time step. As such and for most rainfall events, the effective runoff coefficient throughout an event starts with a given value and increases with time and the amount of fallen rain.

Consequently, the use of an average proportional loss coefficient with the NASHYD sub-model would generate a comparative runoff volume which may not be properly distributed over the duration of the event.

A calibration exercise of NASHYD was undertaken to match simulated and observed peak overflow hydrographs using some of the observed rainfall-runoff events. This resulted in a Unit Hydrograph time to peak 'Tp' of 0.5 and 0.35 hours and a number of linear reservoirs 'N' of 1.8 and 2.4 for Heart's Desire and McFarlane-Pine Glen sewersheds respectively. These values were then used to generate overflow

hydrographs for the selected rainfall events. The peak flows obtained from NASHYD using the effective runoff coefficient given by ANSWAPPS are given in Tables 8 and 9 where they are compared with the measured peak flows. It is seen that in general they compare quite well. This is further illustrated in Figures 32 to 34 where observed and simulated overflow hydrographs are compared for two rainfall events at the Heart's Desire and the McFarlane subdivisions respectively.

From these figures it can be seen that there is a fairly good agreement between the observed and simulated hydrographs.

Table 8: Simulation Results for Heart's Desire Sewershed

Event No.	Storm Duration (hrs/mm)	Total Precipitation (mm)	Maximum Intensity (mm/hr)	Observed			Simulated		
				Runoff Volume (m^3)	Runoff Coefficient (%)	Peak Discharge (l/s)	Runoff Volume (m^3)	Runoff Coefficient (%)	Peak Discharge (l/s)
HDJUN19	13:55	35.9	42.5	69	1.4	7.0	259	5.8	25.0
HDJUL03	18:10	26.4	20.1	35	1.0	3.0	26	0.8	2.0
HDJUL08	8:38	20.1	27	43	1.6	5.2	39	1.6	5.0
HDJUL12	12:28	38.2	24	175	4.1	11.7	139	2.8	9.0
HDJUL19	3:16	20.9	47.9	203	11.2	23.4	250	9.9	23.0
HDSEP8	2:54	10.6	40.2	39	2.8	9.3	16	1.4	2.0

Table 9: Simulation Results for McFarlane-Pine Glen Sewershed

Event No.	Storm Duration (hrs/mm)	Total Precipitation (mm)	Maximum Intensity (mm/hr)	Observed			Simulated		
				Runoff Volume (m^3)	Runoff Coefficient (%)	Peak Discharge (l/s)	Runoff Volume (m^3)	Runoff Coefficient (%)	Peak Discharge (l/s)
PGJUN19	13:55	37.1	53.5	230	6.9	15	149	3.8	21
PGJUL8	7:05	19.3	21.0	230	12.5	14.2	159	7.2	14.0
PGJUL19	4:15	28.0	35.0	294	28.5	36	261	5.7	37
PGAUG04	9:25	62.4	71.8	750	19.5	62	622	9.4	69

7.3 Comments on Model Calibration and Validation

The model's perforated pipe hydraulic components were effectively calibrated with the hydraulic laboratory test data. Through the calibration process it was found that the discharge coefficient for the pipe's perforations varied, with the depth of flow, from a value of say 0.0 to 0.63 . To account for this phenomenon an elliptical interpolating function was incorporated in the model. This improvement significantly increased the model's ability to compute the flows captured by the orifices (see Figure 27).

The overall performance of the ANSWAPPS model was validated through comparisons of simulated effective runoff coefficients with observed values from two grass-swale perforated pipe systems.

Furthermore, the effective runoff coefficients obtained from ANSWAPPS for a typical lot were used with the NASHYD sub-model of OTTHYMO-89 to generate runoff hydrographs for the entire subdivision. It was found that with properly selected Unit Hydrograph time to peak 'Tp' values and number of linear reservoirs 'N' values, very good agreement between simulated and observed data can be achieved.

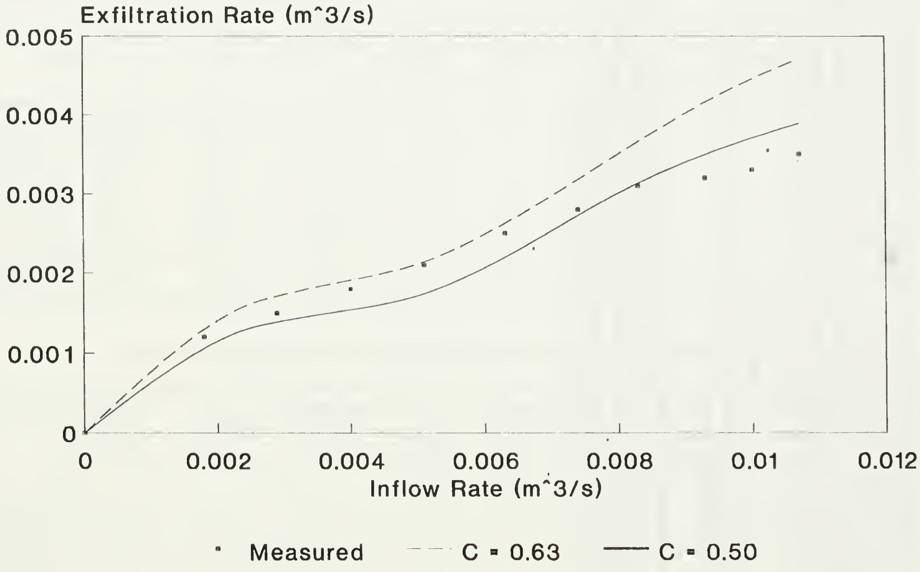

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FIGURE 17. COMPARISON BETWEEN COMPUTED AND OBSERVED EXFILTRATION RATES, (ORIFICE SIZE = 7.9 mm, PIPE SLOPE = 1%)

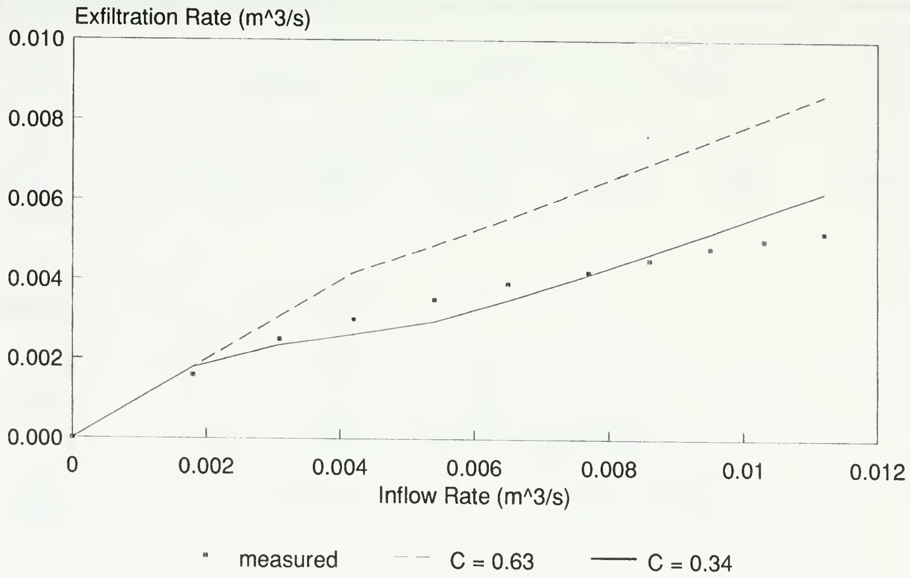

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FIGURE 19. COMPARISON BETWEEN COMPUTED AND OBSERVED EXFILTRATION RATES, (ORIFICE SIZE = 12.7 mm, PIPE SLOPE = 1%)

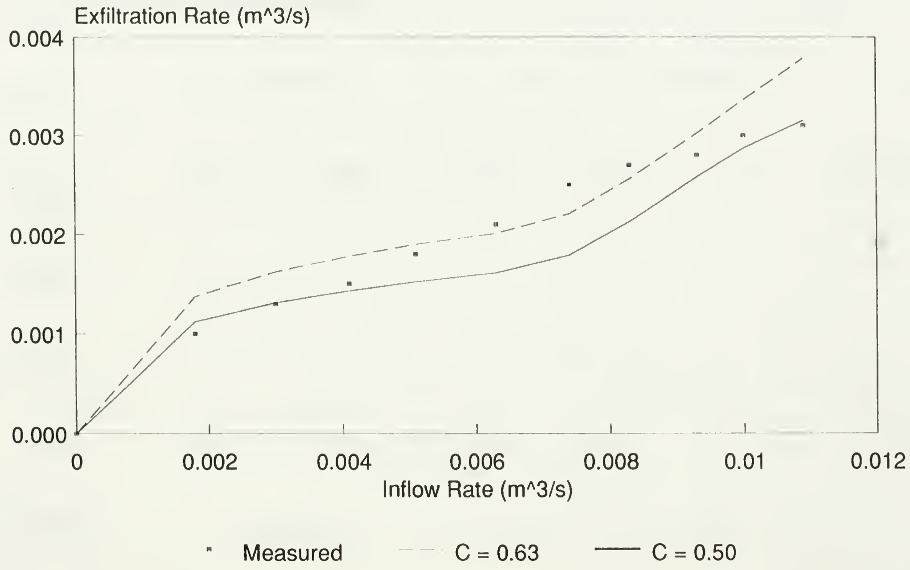

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FIGURE 18. COMPARISON BETWEEN COMPUTED AND OBSERVED EXFILTRATION RATES, (ORIFICE SIZE = 7.9 mm, PIPE SLOPE = 2%)

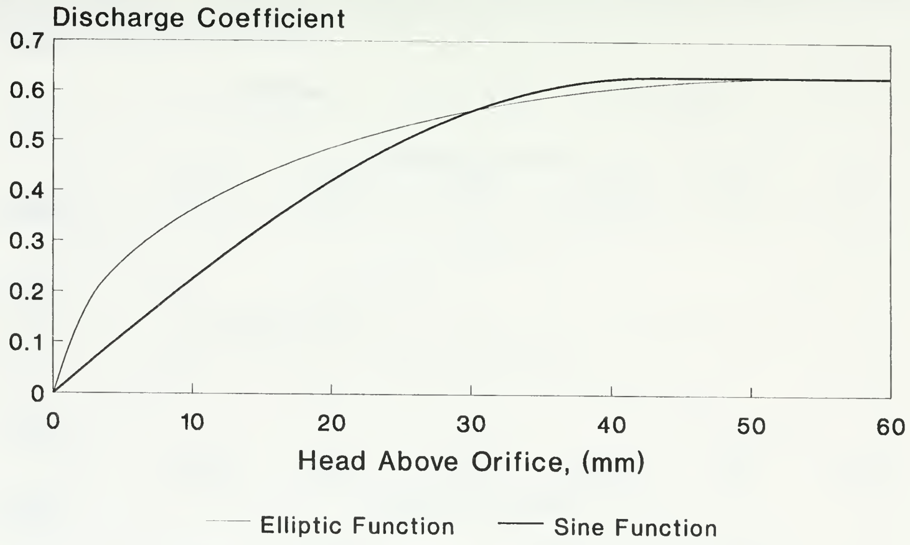

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FIGURE 21. ORIFICE DISCHARGE COEFFICIENT INTERPOLATING CURVES (ORIFICE SIZE = 7.9 mm)

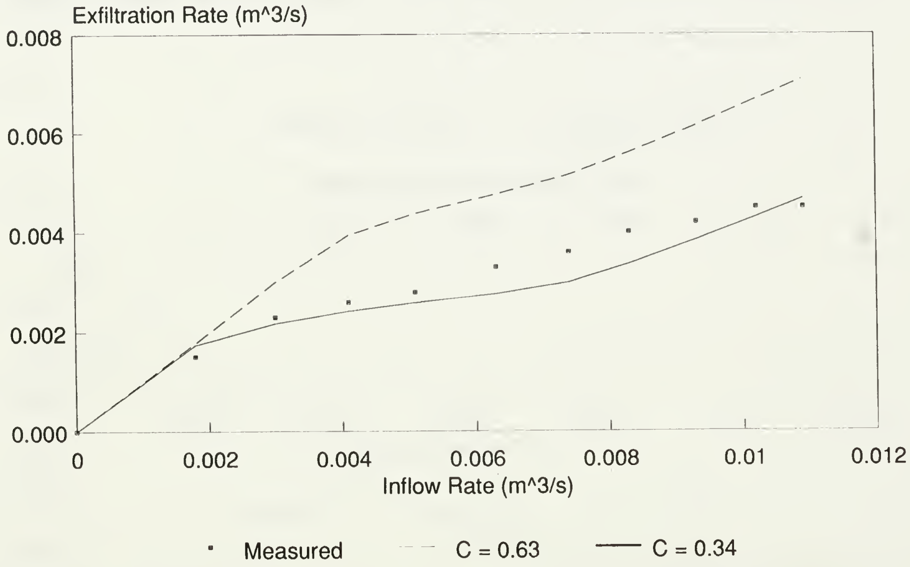

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FIGURE 20. COMPARISON BETWEEN COMPUTED AND OBSERVED EXFILTRATION RATES, (ORIFICE SIZE = 12.7 mm, PIPE SLOPE = 2%)

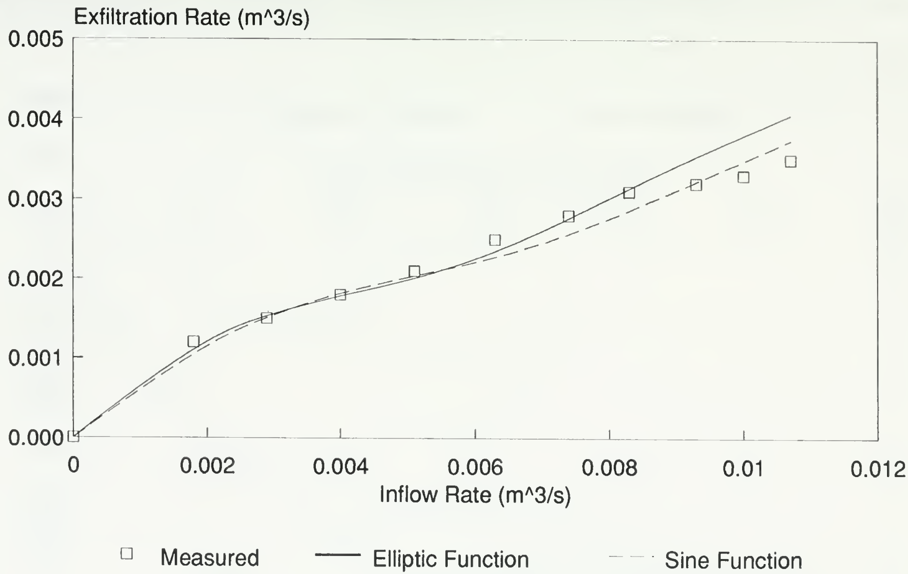

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FIGURE 23. COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT, (ORIFICE SIZE = 7.9 mm, PIPE SLOPE = 1%)

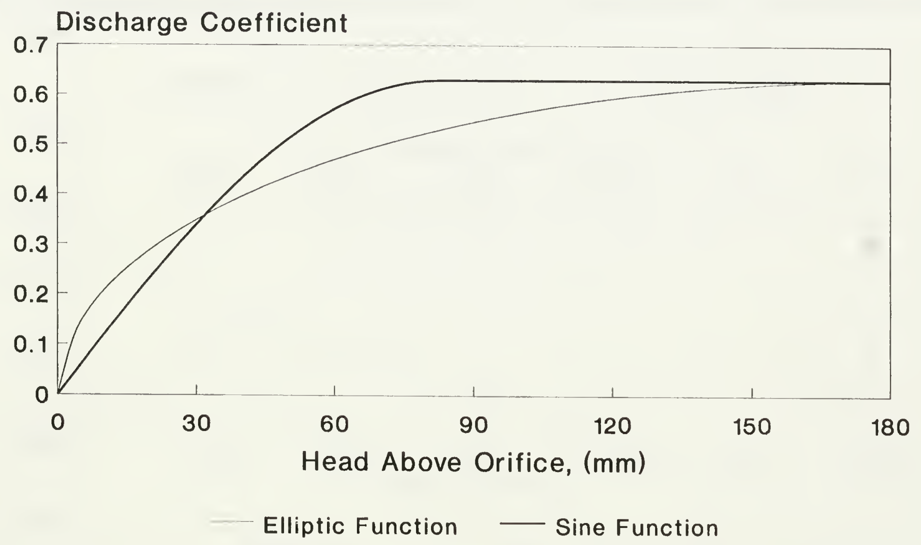

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FIGURE 22. ORIFICE DISCHARGE COEFFICIENT INTERPOLATING CURVES (ORIFICE SIZE = 12.7 mm)

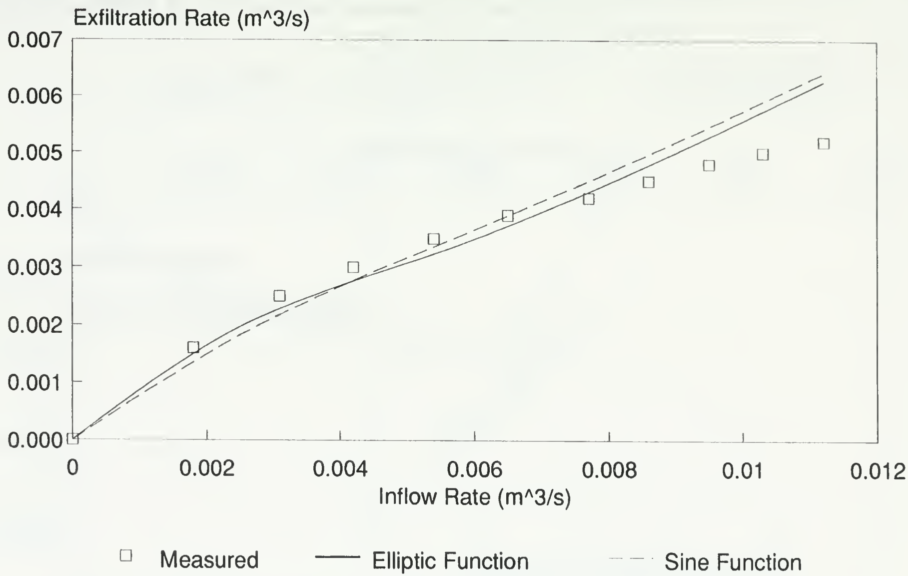

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FIGURE 25. COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT, (ORIFICE SIZE = 12.7 mm, PIPE SLOPE = 1%)

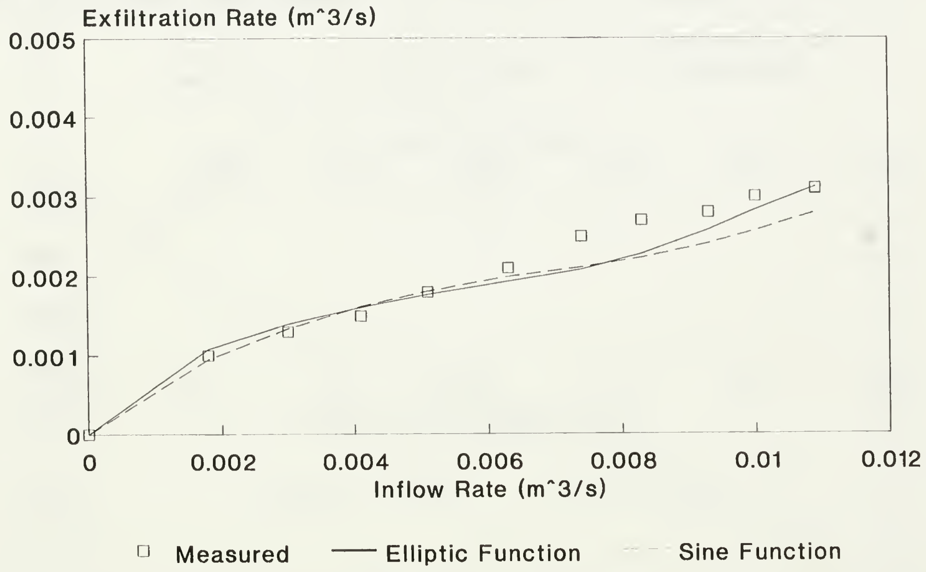

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FIGURE 24. COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT, (ORIFICE SIZE = 7.9 mm, PIPE SLOPE = 2%)

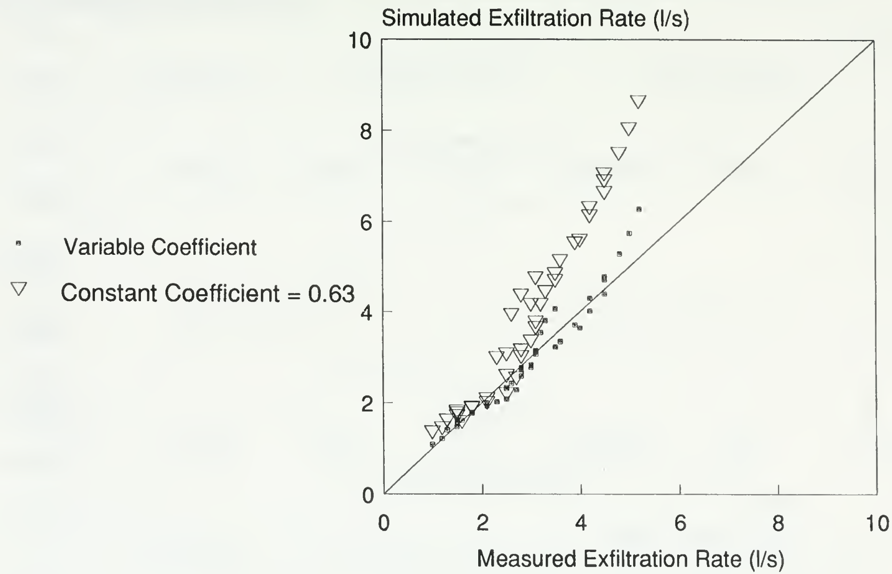

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FIGURE 27. SIMULATED VERSUS OBSERVED CAPTURED FLOW RATES

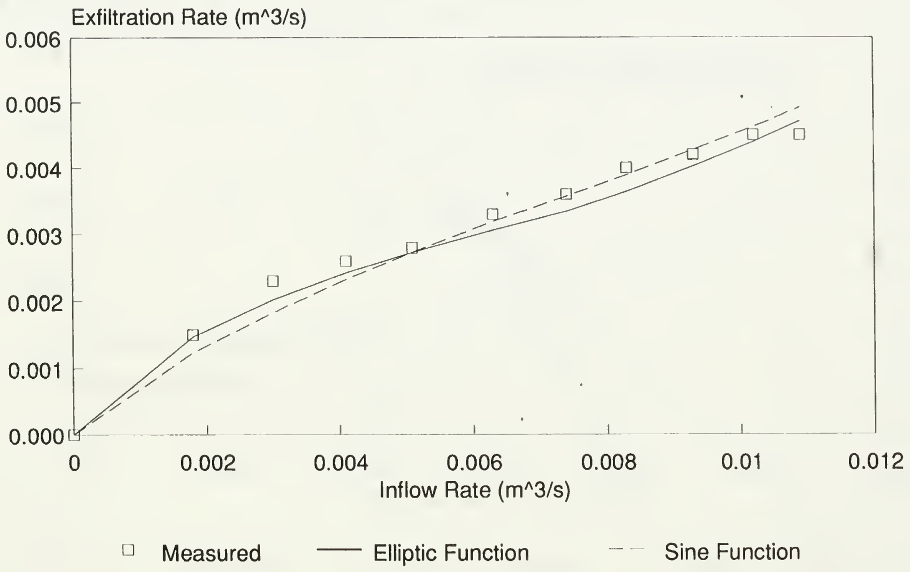

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FIGURE 26. COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT, (ORIFICE SIZE = 12.7 mm, PIPE SLOPE = 2%)

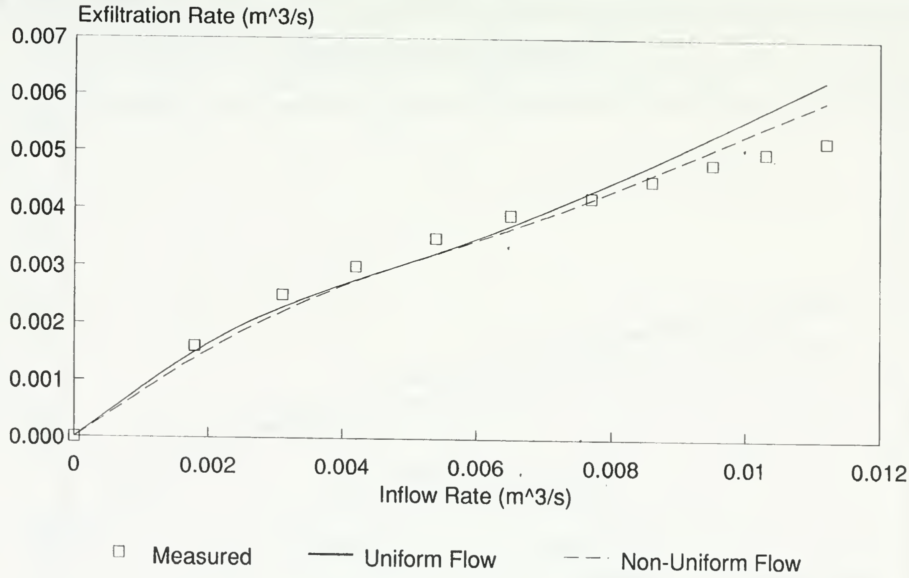

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FIGURE 29. SIMULATED EXFILTRATION RATES USING MANNING'S EQUATION AND GRADUALLY - VARIED FLOW COMPUTATIONS FOR THE WATER SURFACE PROFILE GENERATION.

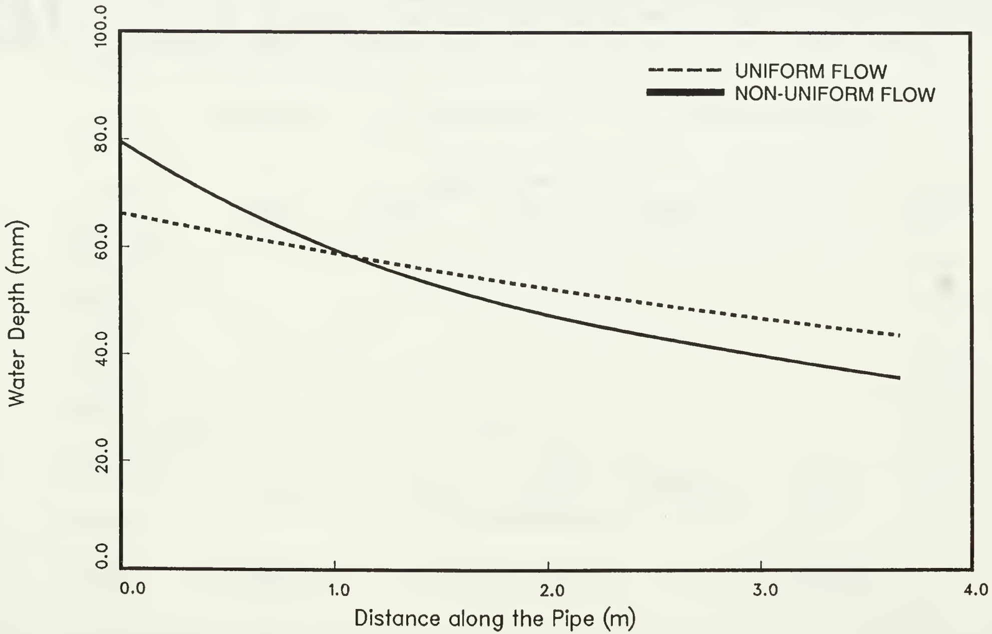

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FIGURE 28. SIMULATED WATER SURFACE PROFILES USING MANNING'S EQUATION AND GRADUALLY - VARIED FLOW COMPUTATIONS.

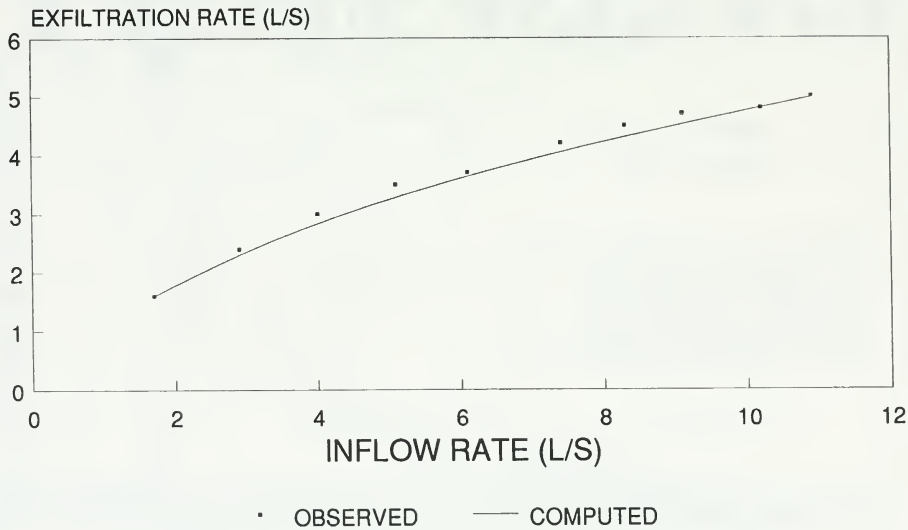

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FIGURE 31. COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT (SLOTS, PIPE SLOPE = 2%).

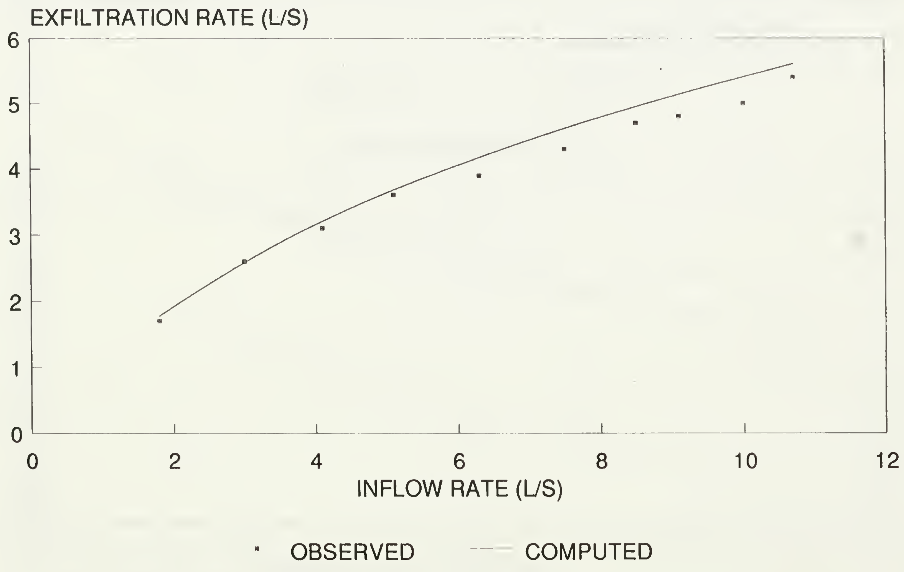

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FIGURE 30. COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT (SLOTS, PIPE SLOPE = 1%).

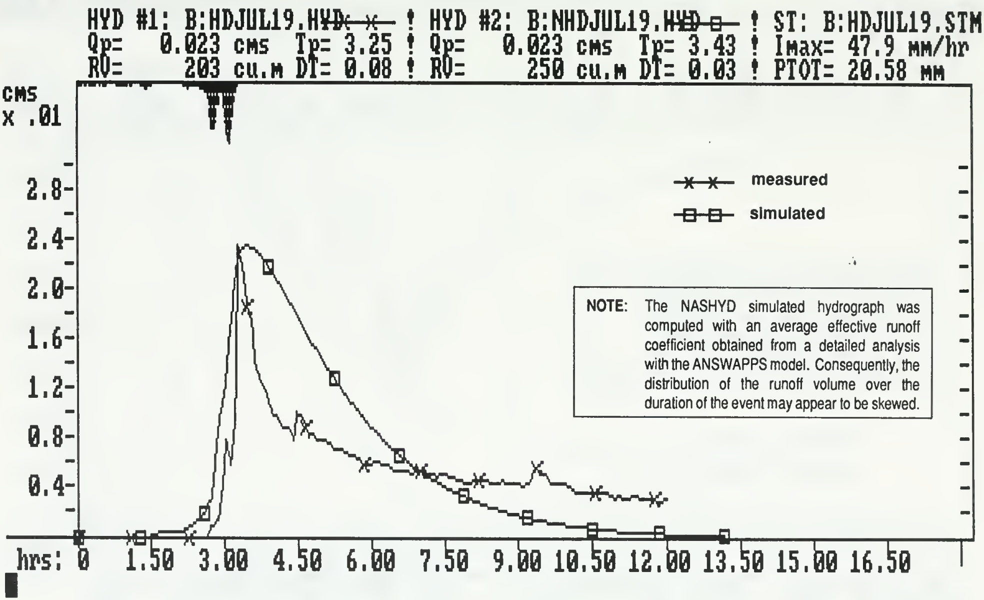

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FIGURE 33. COMPARISON BETWEEN OBSERVED AND SIMULATED OVERFLOW HYDROGRAPHS (HEART'S DESIRE SEWERSHED, EVENT OF JULY 19, 1992).

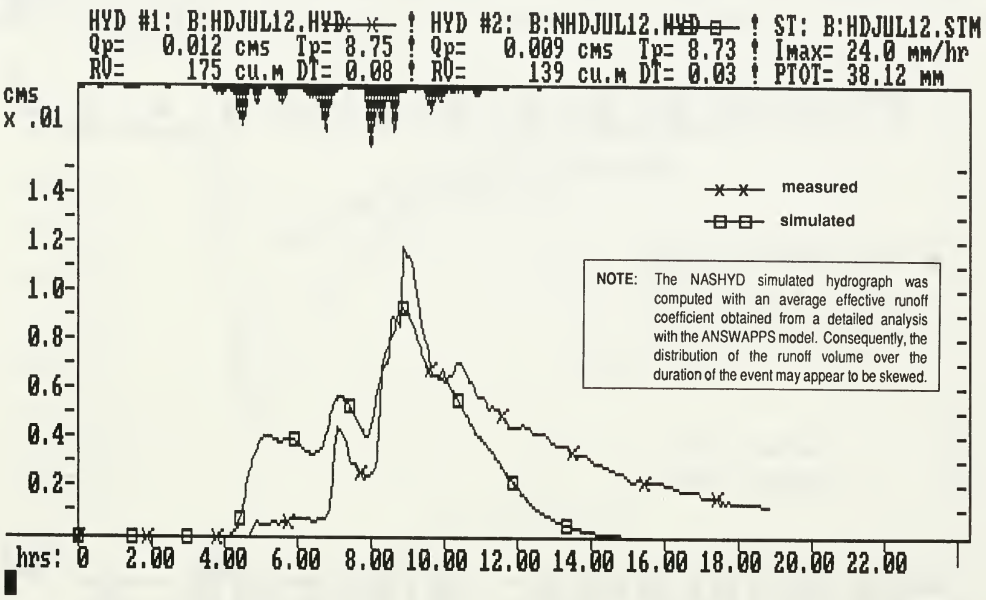

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FIGURE 32. COMPARISON BETWEEN OBSERVED AND SIMULATED OVERFLOW HYDROGRAPHS (HEART'S DESIRE SEWERSHED, EVENT OF JULY 12, 1992).

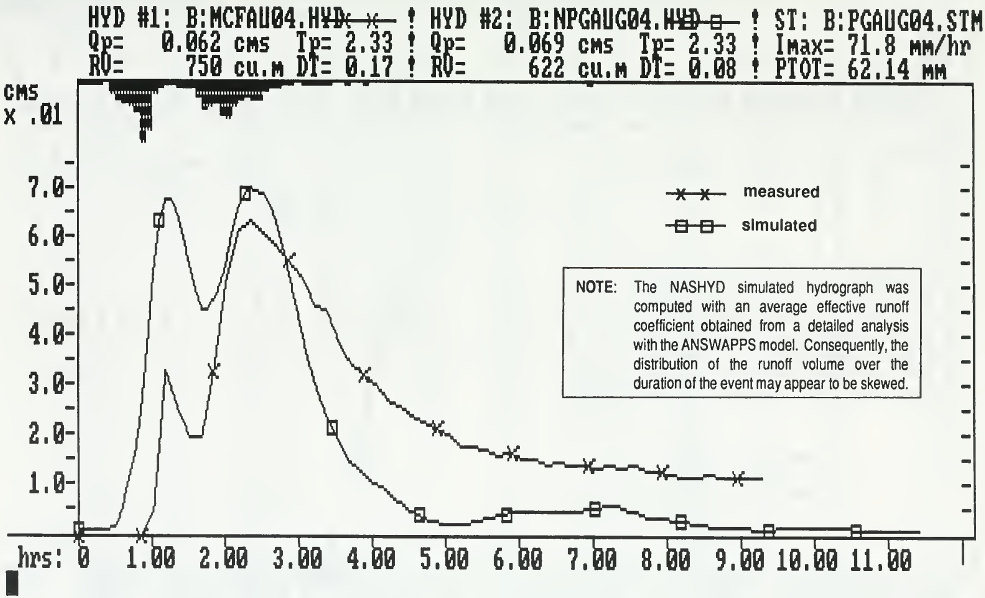

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FIGURE 35. COMPARISON BETWEEN OBSERVED AND SIMULATED OVERFLOW HYDROGRAPHS (MCFARLANE - PINE GLEN SEWERSHEDS, EVENT OF AUGUST 4, 1992).

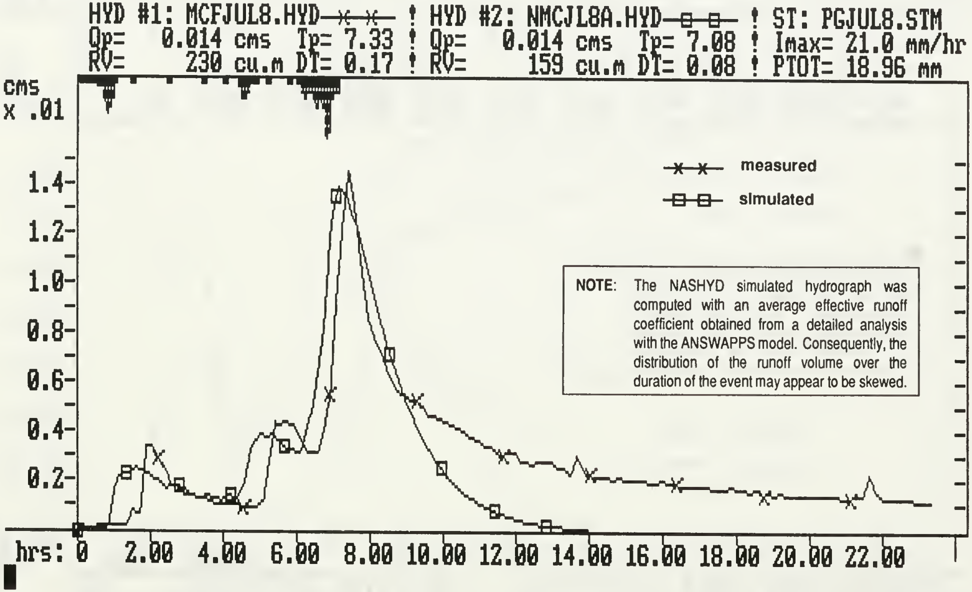

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FIGURE 34. COMPARISON BETWEEN OBSERVED AND SIMULATED OVERFLOW HYDROGRAPHS (MCFARLANE - PINE GLEN SEWERSHED, EVENT OF JULY 8, 1992).

The water in the trench below the pipe is infiltrated in the native soil based on the trench's dimensions and soil characteristics. A reduction factor can be used to account for long term clogging.

4) **Calculate the system overflow.**

Overflow from perforated pipe systems consists of two components: i) pipe overflow, and ii) trench overflow.

The first component (pipe overflow) is made up of the pipe flow which is not captured by the pipe's orifices.

The second component (trench overflow) is from the water which is conveyed in the trench and does not have sufficient time to infiltrate over the length of the trench as the water is slow moving downstream.

ANSWAPPS, which was calibrated and validated in the previous Sections, is further used to investigate the sensitivity of various design parameters. Similar exercises to the ones described below could help in the development of design guidelines.

The performance of a theoretical system was assessed by arbitrarily changing some of its design parameters. The design parameters which were varied are:

- lot size and imperviousness;
- grass swale geometry and infiltration capacity;
- top soil layer thickness and its permeability;
- pipe length, diameter, and slope;
- size, number and configuration of orifices around the pipe circumference;
- trench dimensions; and
- native soil infiltration capacity.

Since most stormwater quality guidelines could inherently be met if the runoff of a 25 mm (1 inch) storm is retained and infiltrated on site, a typical one-inch 4 hour Chicago storm distribution was used to simulate the surface runoff.

8.1 Effects of Lot Imperviousness on Trench Storage Requirements

In this application the model was used to simulate the minimum trench depth required to capture runoff from the one-inch storm for different native soils (below the trench) and different lot imperviousness ratios. Other data used in the simulations are given as follows:

- lot size = 0.1 ha

8.0 MODEL APPLICATION

The ANSWAPPS model applies the theory and modelling approach presented in Section 5. As such, the model which is listed in Appendix F undertakes the following tasks:

1) **Calculate or obtain a previously computed surface runoff hydrograph. This can be accomplished in two ways:**

   a) read from a disk file a hydrograph previously simulated by an external method.

   b) read from a disk file a rainfall hyetograph and generate a surface runoff hydrograph based on lot size and imperviousness. In this case, the initial abstraction is first depleted and a proportional loss coefficient based on imperviousness is used to simulate the hydrograph

2) **Convey the surface runoff over the grass swale and determine the amount of runoff captured by the catchbasin.**

   To do this, the model uses the surface runoff obtained in (1) and distributes it evenly along the sides of the swale over which some infiltration is calculated based on soil characteristics.

   The excess flow is then conveyed along the centre of the swale towards the catchbasin which is assumed to be located at the low point. Over this flow length, the infiltration is again calculated based on soil characteristics.

   The excess flow from the above computations is assumed to be captured by the catchbasin.

3) **Determine how much of the pipe inflow is captured by the pipe's orifices, how much water is retained in the storage below the pipe and infiltrated into the native soils.**

   The total pipe inflow is made up of the flow captured by the catchbasin plus the flow infiltrated in the swale delayed by a time based on depth of pipe and soil characteristics.

   The flow captured by the pipe's orifices is based on flow depth, orifice size and number, and conditions of the trench how much water is retained in (granular bedding below the pipe).

- surface initial abstraction = 2.0 mm
- lot imperviousness= varied from approx. 15% to 55%
- swale/pipe length = 25.0 m
- swale side slope = V:1, H:25
- swale width = 5.0 m
- swale Manning's roughness coefficient = 0.3
- pipe diameter = 0.3 m
- pipe Manning's roughness coefficient = 0.012
- swale/pipe longitudinal slope = 1%
- diameter of pipe perforations = 1.27 cm
- spacing between rings of perforation = 5.0 cm
- number of perforations around the pipe's circumference = 8
- trench width = 0.75 m
- void ratio of the granular material in the trench = 0.35

The simple coefficient method provided in the model was used to simulate the surface runoff hydrograph. It is reminded that this method was only incorporated for convenience and that any hydrograph derived with other methods or models and be used.

In our simulations the following equation was used to estimate the surface runoff coefficient from the lot imperviousness:

$$C_l = 0.9I_l + 0.15\ (1 - I_l) \qquad \textbf{(18)}$$

where I_l = lot imperviousness ratio and
C_l = lot runoff coefficient.

Equilibrium or minimum infiltration rates for the different soils used were taken from the literature and are given in Table 7 in the previous Section.

Two types of top soil (sandy loam and silty loam) were considered having infiltration rates of 3 cm/hr and 1 cm/hr respectively. The results are shown graphically in Figures 36 and 37 respectively. Both figures show that the minimum required trench depth increases with an increase in imperviousness. This result was anticipated as an increase in imperviousness implies an increase in the volumetric runoff coefficient and therefore an increase in runoff volume.

Soils with lower infiltration rates, which belong to the SCS soil group D, were not considered because they are not recommended for infiltration practices. Examining again Figures 36 and 37, it is apparent that the minimum trench depth required to retain and infiltrate the runoff from a 25 mm (1 inch) storm is only marginally sensitive to a change in native soil infiltration capacity. In fact, even though the

maximum difference between their corresponding minimum trench depths is usually less than 10%.

8.2 Effects of Trench Width and Depth on System Performance

The effect of varying the trench width on system performance was also investigated. In this simulation, loam was selected as top soil while the native soil was sandy loam. The average lot imperviousness ratio was 35% and the remaining data were the same as given previously. Different combinations of trench width and depth values, which give a constant trench volume, were considered. The corresponding overflow volumes are shown numerically in Table 10 and graphically in Figure 38.

Table 10: Effect of Trench Dimensions on System Performance

Trench Width (m)	Trench Depth (m)	Overflow (m^3)
0.5	0.490	2.806
1.0	0.245	2.716
1.5	0.163	2.556
2.0	0.123	2.426

The results shown in the above table indicate that the wider the trench, the smaller is the overflow volume and therefore the larger is the captured runoff volume. This is mainly explained by the fact that a wide trench provides a greater infiltration area, which enhances the system's performance. It is important to note at this stage that the infiltration area was taken as the area of the sides and the bottom of the trench multiplied by a factor to account for clogging. This factor, which was taken from Table 3 based on an assumed design life of 20 years, was 65%. Although some literature suggest that clogging mainly occurs at the bottom of the trench, it was conservatively assumed in the above example to also occur on the sides.

8.3 Sizing the Grass Swale

Although the emphasis on the purpose of the grass swale-perforated pipe system is mainly to retain and infiltrate runoff from small storms, the physical characteristics of the grass swale should be based on the design storm flow of the minor drainage system (e.g. 5 year storm). That is the grass swale should be able to contain and convey the design peak flow. Figure 39 shows the minimum swale width required to confine a range of peak flows for different swale side slopes. The figure was produced based on the following data:

- Lot size = 0.1 ha
- Swale longitudinal slope = 1%
- Swale Manning's roughness coefficient = 0.3

As an example, using the Nepean 5-yr Chicago storm and a lot imperviousness of 25% a lot peak flow of approximately 5 l/s would be generated. Depending on the swale's side slopes, the swale's width would have to be between 4 and 7 m (see Figure 39).

8.4 Design Storm Runoff Volumes for Existing Systems

Simulations with the ANSWAPPS model were performed to compute overflow volumes and volumetric runoff coefficients for average lot sizes and pipe lengths. The analyses were conducted with synthetic one-year, two-year and five-year, 3 hour Chicago storms derived from the City of Nepean IDF curves. The input parameters were based on the physical characteristics of the two monitored Nepean subdivisions (Heart's Desire and McFarlane-Pine Glen) which are described in other Sections. Results of these simulations are shown in Tables 11 and 12 for both subdivisions, respectively.

Table 11: Computed Overflow Volumes and Runoff Coefficients For Heart's Desire Sewershed (Synthetic Storms) (Lot Size = 0.20 ha, Imp = 25%)

Storm	Total Precipitation (mm)	Total Surface Runoff (m^3)	Overflow (m^3)		Runoff Coefficient (%)	
			with trench flow	without trench flow	with trench flow	without trench flow
One-year	25.0	11.16	3.72	3.19	7.4	6.4
Two-year	33.4	15.19	6.83	6.25	10.2	9.4
Five-year	46.0	21.26	11.11	10.44	12.1	11.3

Table 12: Computed Overflow Volumes and Runoff Coefficients For McFarlane-Pine Glen Sewershed (Synthetic Storms) (Lot Size = 0.26 ha, Imp = 25%)

Storm	Total Precipitation (mm)	Total Surface Runoff (m^3)	Overflow (m^3)		Runoff Coefficient (%)	
			with trench flow	without trench flow	with trench flow	without trench flow
One-year	25.0	14.700	2.55	2.31	3.9	3.5
Two-year	33.4	20.014	6.13	5.85	7.1	6.7
Five-year	46.0	28.024	10.77	10.44	9.0	8.7

By comparing Tables 11 and 12, it can be seen that runoff coefficients at Heart's Desire are higher than those at McFarlane-Pine Glen Subdivision. This is mainly explained by the relatively lower native soil infiltration rates in Heart's Desire Subdivision.

The simulations also indicate that the prevention of trench flow would further reduce the resulting effective runoff coefficient by as much as 20% to 15% for a one-year storm.

8.5 Range of Applicability and Model Limitations

The model developed for the ANalysis of grass SWales And Perforated Pipe Systems (ANSWAPPS) was validated against laboratory and field measurements. Furthermore, the model was tested with various theoretical scenarios and was found to give consistent results.

However, in view of the model's computational procedures and theoretical background the model does have limitations and should, like any other model, be used with care and good engineering judgement.

In general, the model should mainly be used for short pipe lengths (less than 150 m), the selected computational time step should be between 1 and 2.5 minutes, simulated flows to be conveyed in the pipe should not exceed the pipe capacity, the pipe and swale slopes must be positive (ie. greater than zero), etc.

The complete listing of the ANSWAPPS computer model is given in Appendix F with a brief User's Manual.

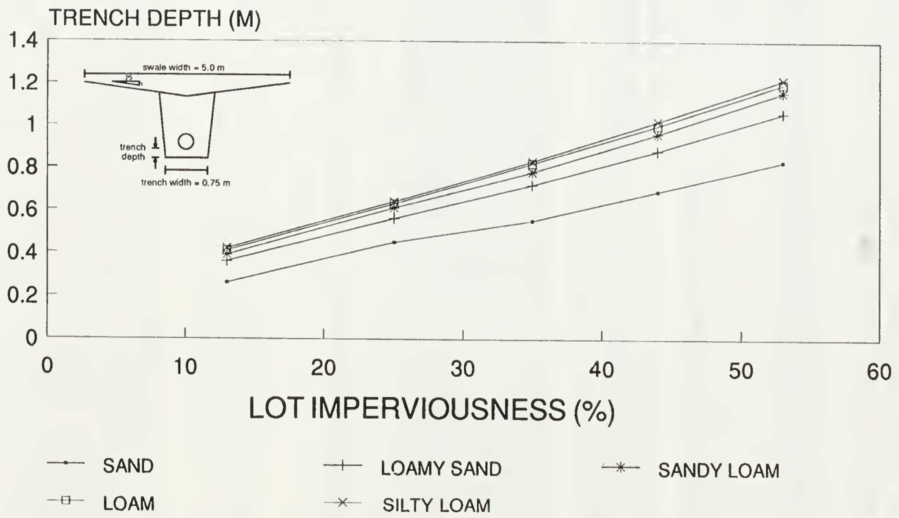

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FIGURE 36. MINIMUM TRENCH DEPTHS REQUIRED TO CAPTURE RUNOFF FROM THE ONE-INCH CHICAGO STORM (TOP SOIL = LOAM).
(see Section 8.1 for other swale, pipe and trench data)

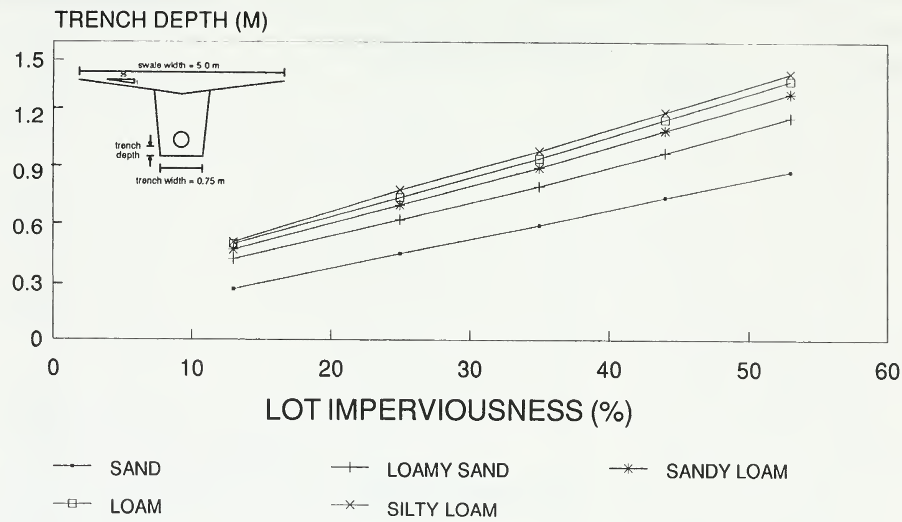

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FIGURE 37. MINIMUM TRENCH DEPTHS REQUIRED TO CAPTURE RUNOFF FROM THE ONE-INCH CHICAGO STORM (TOP SOIL = SILTY LOAM).

(see Section 8.1 for other swale, pipe and trench data)

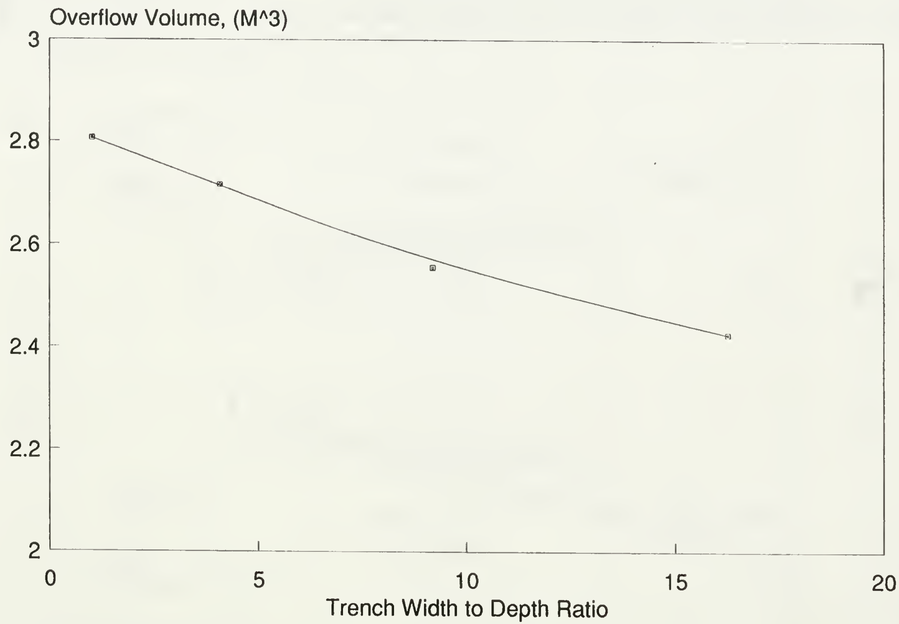

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FIGURE 38. THE EFFECT OF THE TRENCH WIDTH TO DEPTH RATIO ON OVERFLOW VOLUME.

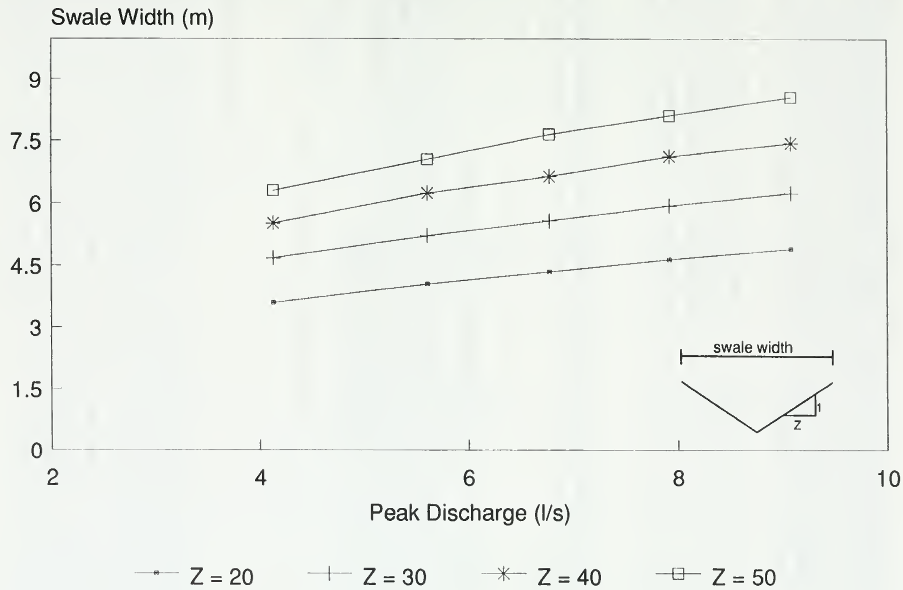

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FIGURE 39. MINIMUM SWALE WIDTHS REQUIRED TO CONFINE RUNOFF FROM THE FIVE-YEAR CHICAGO STORM.

9.0 STORMWATER QUANTITY MONITORING

According to the terms of reference, only one grass swale-perforated pipe system was to be monitored. However, funds under a separate contract with the City of Nepean (for work required by MOE) were provided to monitor two grass swale-perforated pipe systems for stormwater runoff quantities and E.Coli concentrations. In view of this, it was suggested that in order to provide additional useful information, the monitoring program be expanded to include a conventional concrete sewer system.

Continuous monitoring of rainfall, stormwater quantity and quality (discussed in Section 10) was conducted within three mature residential subdivisions in the City of Nepean. Two of these subdivisions (Heart's Desire & McFarlane-Pine Glen) had grass swale-perforated pipe systems while the third one (Amberwood Phase 1) had a conventional concrete pipe system. The location of the three sites are shown on Figure 40.

The purpose of the flow monitoring program was to evaluate the general performance of perforated pipes and compare it with the performance of a conventional concrete storm sewers.

9.1 Brief Description of the Monitored Sewersheds

The main characteristics of the three monitored sewersheds are listed in Table 11. The McFarlane-Pine Glen and Heart's Desire sewersheds have grass swale-perforated pipe system according to the Nepean design standards while the Amberwood-Phase 1 system is serviced by a conventional type system.

Table 13: Main Characteristics of the Monitored Sewersheds

Characteristic	Nepean Perforated Pipe System (NPPS)		Conventional
	Heart's Desire	McFarlane-Pine Glen	Amberwood Phase 1
Land Use	Residential	Res/Comm	Residential
Sewershed Area	13.64 ha	10.02 ha	12.08 ha
Impervious Area Draining Directly to Grassed Swales	15%	17%	25%
Total Imperviousness	25%	25%	35%
Number of Lots	70	39	82
Average Lot Size	0.20 ha	0.26 ha	0.13 ha
Total Pipe Length	3045 m	2685 m	1204 m
Relative Pipe Length	223 m/ha	268 m/ha	100 m/ha
Type of Underlying Soils	Silty Till	Sand/Silty Till	n.d.
Age of Subdivision	33-36 yrs	37 yrs	20 yrs
Age of Perforated Pipe System	1-6 yrs	3-6 yrs	n.a.

9.1.1 McFarlane-Pine Glen

The McFarlane-Pine Glen NPPS (Figure 41a) contains some reaches along McFarlane Road which are relatively deep (2.5-3.5 metres below surface). As a result of the high local water table, this part of the system is subject to continuous baseflow caused by groundwater infiltration. Some baseflow was observed in the shallower portions of the system during the spring and early part of the summer as a result of the increased water table elevation associated with spring snowmelt. It should also be mentioned that about 25% of the impervious area (Drummond's Gas Station) in McFarlane-Pine Glen is directly connected to the perforated pipes and does not benefit from the buffer effects of the vegetative swale.

Soil investigations in the McFarlane-Pine Glen NPPS indicated that the area is overlain by a 1.5 to 2.0 metre layer of fine silty sand to sandy silt. These layers are underlain in turn by less permeable silt and clayey silt. The upper sandy soils are favourable for infiltration, however, their infiltration capacity is somewhat limited by the underlying silty layers. Soil investigation data is provided in Appendix G.

9.1.2 Heart's Desire

The Heart's Desire NPPS sewershed is outlined in Figure 41b. Because of the somewhat accentuated topography, system depths vary up to a maximum of 2.0 metres below surface. Although baseflows were not observed at the system outlet where measurements were taken, some baseflows were seen in some of the deeper parts of the system.

Soils overlying the Heart's Desire sewershed consist mainly of sandy silt till. These soils are well compacted and are less suitable for infiltration than the soils at McFarlane-Pine Glen. Soil investigation data is provided in Appendix G.

9.1.3 Amberwood Phase 1

The Amberwood-Phase 1 conventional system sewershed is outlined in Figure 41c. In general, the system is deeper than the NPPS. A small, continuous, year-round baseflow as a result of groundwater infiltration was observed at the outlet.

9.2 Monitoring Equipment and Data Processing

For all three systems, flows were measured at the first upstream manhole from the outlet (Figures 42a, b, c). The flows for the NPPS were measured using triangular, sharp-crested weirs designed and calibrated in-house by PWA staff. Upstream head levels were recorded using a stilling well/float assembly connected with a Lakewood System's data logger. A typical monitoring installation is detailed in Figure 43. Flows for the conventional concrete pipe system were determined using a theoretical stage-discharge relationship based on pipe diameter, pipe slope, Manning's n = .013 and water level. Water levels were measured in a similar fashion to the NPPS.

Local rainfall was measured at two locations using tipping-bucket rain gauges connected to data loggers. Rainfall for the Heart's Desire subdivision was recorded at a private residence located at 61 Cortleigh Drive. Rainfall for the McFarlane-Pine Glen and Amberwood-Phase 1 subdivisions was recorded at the Nepean Hydro building located at 1970 Merivale Road.

Typical hyetographs and hydrographs are shown in Figure 44. The complete set of hyetographs and hydrographs as well as summary data sheets are included in Appendices H and I.

Rainfall and runoff data were first processed to determine the initial abstraction, volumetric runoff coefficients and runoff coefficients using the Rational Method. Comparisons between measured and theoretical values were made for each site. These results are presented in the following sections.

9.2.1 Rainfall Characteristics

Rainfall data were recorded from June 15th to September 20, 1992 in a relatively rainy year (Appendix H). During 97 days of observations, 28 and 31 rainfall events were recorded at the Pine Glen and Heart's Desire stations respectively. This corresponds to one rainfall event every 3.5 days at McFarlane-Pine Glen and Amberwood-Phase 1 sewershed and one every 3.1 days at Heart's Desire sewershed. Comparisons with previous years monthly and yearly extremes are given in Table 14.

Table 14: Summer Precipitation Data for 1992

Month	Monitoring By PWA Summer * 1992 (mm)		Average ** 1939-1987 (mm)	Monthly Extremes ** 1939-1987 (mm)		Yearly Extremes ** 1961-1987 (mm)	
	Pine Glen	Heart's Desire		Most	Least	Most (1981)	Least (1964)
June	57.5	58.9	73.4	174.8	18.3	155.1	31.5
July	173.7	185.3	86.9	186.4	28.1	57.7	91.7
August	170.5	157.0	88.4	181.4	8.4	180.5	34.8
September	44.8	51.7	79.3	173.0	13.2	144.0	29.5
Totals	446.5	452.9	458.5	1039.7	99.2	774.5	272.6

* Period June 15 to September 20, 1992 inclusive
** Source: Delcan Corporation, 1988

Figure 45 shows the percentage of rainfall events and runoff volumes[1] of different magnitude. It is seen that 60% of all rainfall events recorded during the monitoring period were less than 12 mm.

The most significant events during the monitoring period were observed on July 17th and August 4th. For the 60 to 180 minute time intervals, the July 17th storm recorded at Pine Glen was equivalent to that with a 2 year return period (see Figures 46 and 47 for the hyetograph and comparison with the IDF curve). Similarly, the August 4th storm exhibited a 2 year return period for the 30 and 60 minute intervals and a 25 year return period for the 60 to 180 minute intervals.

9.2.2 Initial Abstraction and Number of Overflows

For a perforated pipe system an overflow can be defined as a volume of water discharged from the system outlet to a receiving body of water. Some of the rainfalls did not generate any overflows at the outlet of the NPPS system. Even when an overflow was observed, it started only if the cumulative precipitation volume exceeded a given value, termed the Total Initial Abstraction (TIA). TIA has two components:

1) Surface Initial Abstraction (SIA) in the grassed and paved areas. In general this amount is estimated to be in the order of 1.5 mm to 2.5 mm.

2) Underground Storage Initial Abstraction (USIA) resulting from the storage capacity in the gravel bed below the pipe invert.

A preliminary estimate of the TIA can be obtained as follows:

> For the two investigated NPPS, the gravel bed depths varied from 10 to 15 cm. Using the total length of pipe trench for each site and an assumed porosity of 35% for the granular fill below the pipe, a theoretical USIA range of 1.8 to 2.7 and 2.0 to 3.1 mm may be calculated for Heart's Desire and McFarlane-Pine Glen, respectively. The Total Initial Abstractions including SIA thus range from 3.3 to 4.2 mm for Heart's Desire and from 3.5 to 4.6 for McFarlane-Pine Glen.

These rough estimates were first compared with the minimum total rainfall for which an overflow was observed. For the NPPS this value varied from 1.7 mm for McFarlane-Pine Glen to 4.8 mm for Heart's Desire. It is reminded that for McFarlane-Pine Glen the storage in the gravel bed for the deeper reaches is not

1 The total runoff volume less baseflow measured 24 hours after the end of the rainfall event.

available because of the higher groundwater table which was observed. For the conventional system it is, as expected, less than 1 mm.

The TIA values were also determined directly albeit approximately from the comparison of hyetographs and hydrographs. An error with this approach is unavoidable due to lack of perfect synchronization. Figure 48 shows that TIA is of the same order of magnitude as the theoretical value found on the basis of bed storage.

Figure 45 shows the percentage number of events for which there was zero runoff (no overflow) for specific rainfall intervals. For rainfall events less than 6 mm 92% did not generate runoff at Heart's Desire while 25% did not generate runoff at McFarlane-Pine Glen. All events at Amberwood generated runoff.

9.2.3 Peak Flows

Rational Method runoff coefficients (Figures 49a and b) for measured peak flows (Appendix H) were compared with the "theoretical" values. A 15 minute time of concentration gives reasonable values for the conventional system. For the NPPS the inlet time is greater because of smaller velocities in the swale and the time of concentration selected is 30 minutes (which is also consistent with the time to peak obtained in Section 7). Theoretical C values were calculated for each subdivision on the basis of imperviousness as shown in Table 15 below. Note that measured runoff coefficient for Amberwood Phase 1 may be greater than theoretical values because of the unusually wet nature of the monitoring period.

Table 15: Measured and Theoretical Peak Flow 'C' Values for NPPS and Conventional Systems

System	Tc (min)	C observed*	Theoretical C for Conventional System $C_{theor} = 0.9(I) + 0.15(1 - I)$	
			Roofs Connected	Roofs Disconnected
Heart's Desire	30	0.035	0.34	0.26
McFarlane	30	0.05	0.34	0.28
Amberwood	15	0.45	0.41	0.34

* Based on measurements from June 15 to September 20, 1992

It is seen that because part of the McFarlane-Pine Glen system is subject to groundwater infiltration, the 'C' value for this sewershed is higher than that at Heart's Desire. This was confirmed by a more detailed analysis which accounted for areas contributing to the deeper and shallower reaches independently.

In order to show the significant difference between perforated and conventional storm sewer systems peak flows determined from intensities taken from a two year IDF curve for each NPPS were compared with theoretical peak flows calculated on the basis of directly connected imperviousness. This comparison is shown in Table 16.

Table 16: Comparison of Peak Flows for Conventional and Perforated Pipe Systems

System	System Type	Q_{peak} (cms)	% Reduction
Heart's Desire	NPPS (Tc = 30 min, C = 0.035, I_{30} = 38 mm/hr)	0.050	93.2%
	Theoretical Conventional (Tc = 15 min, C = 0.31, I_{15} = 63 mm/hr)	0.741	
McFarlane-Pine Glen	NPPS (Tc = 30 min, C = 0.050, I_{30} = 38mm/hr)	0.053	90.6%
	Theoretical Conventional (Tc = 15 min, C = 0.32, I_{15} = 63 mm/hr)	0.561	

It was found that observed peak flows for the NPPS were reduced by 93.2% and 90.6% for Heart's Desire and McFarlane-Pine Glen, respectively, when compared with peak flows that could be expected based on theoretical 'C' values. However, such conclusion would require additional investigations such as measurements taken during larger storms.

9.2.4 Runoff Volumes

Runoff volumes are very important for the assessment of reduction in pollutant loading. Runoff volumes for individual events were computed for the duration of the storm as well as for 3, 6, 12 and 24 hours after the rainfall had stopped. Total runoff volumes were also calculated until the runoff stopped or until the start of a subsequent storm or if a baseflow was present before the storm then the total runoff volume is calculated until the flow returns to pre-storm baseflow conditions. For each event of the monitoring period, these values are listed in the Tables of Appendix H.

The total seasonal runoff volumes measured throughout the duration of the monitoring period and are presented in Table 17.

A comparison of total seasonal runoff volumes per sewershed area shows that the total seasonal runoff for the NPPS is 2.7 and 12.0 times smaller (McFarlane-Pine Glen and Heart's Desire) than for the conventional Amberwood Phase 1 system. This is very important in assessing the recharge capability and favourable effect of the NPPS on the hydrologic budget and pollutant loading (Figure 50). *The higher runoff volume at McFarlane-Pine Glen is mainly attributed to the loss of storage in the gravel bed which is below the groundwater table in the deeper reaches of the system.*

Table 17: Seasonal Runoff Volumes for NPPS and Conventional Systems

	NPPS		Traditional
	Heart's Desire	McFarlane	Amberwood
Total Seasonal Rainfall (mm)	452.9	398.6 *	446.5
Total Runoff Volume (m^3)	1996.1	5635.8	20682
Total Runoff (mm)	14.6	56.3	171.2
Runoff Volume for Rainfall Events Less Than 12 mm (m^3)	65.1	419.6	2083

* Rainfall volume corrected to account for runoff monitoring equipment failure 28/06/92 to 7/07/92

9.2.5 Event Volumetric Runoff Coefficient

Volumetric event runoff coefficients were calculated for the three systems as follows:

$$Cv = \frac{\text{Total event volume (24 hours)}}{\text{Total Precipitation}}$$

Cv is usually a function of imperviousness (I) ratio and precipitation (P). This relationship is shown by plotting Cv/I vs P as shown in Figure 51a and Figure 51b. It is seen that for P > 5 mm, Cv/I has less variability as shown in Table 18.

Table 18: Ratios of Volumetric Runoff Coefficients to Imperviousness

System	Cv observed	$\frac{Cv}{I}$
NPPS Heart's Desire	0.06	0.35
NPPS McFarlane-Pine Glen	0.17	1.0
Conventional Amberwood	0.40	1.6

It is seen that for the Heart's Desire system, the volumes of runoff per event are reduced on average by 67% as compared with the McFarlane-Pine Glen system. Higher runoff volumes observed at McFarlane-Pine Glen may be attributed to a combination of system specific parameters including continuous baseflow (which results in loss of trench storage capability as well as loss of infiltration capacity) and directly connected impervious runoff areas.

9.2.6 Exfiltration Rates and Trench Storage Durations

For comparison purposes, the durations of exfiltration of the volumes of water stored in the gravel bed below the perforated pipe were determined for the McFarlane-Pine Glen and Heart's Desire sites. Durations of exfiltration were calculated by dividing the true depth of water (trench depth below pipe invert x porosity) by the rate of infiltration 'K_{ns}', for the native soil below the trench. Depths of water below the pipe are dependent upon the thickness of granular fill which varies between 0.10 and 0.15 metres. Selected 'K_{ns}' values and duration of exfiltration are given in Table 19.

Consideration must be given to this parameter when designing exfiltration trenches in order to minimize the number of overflows.

Table 19: Duration of Release from Gravel Bed Storage

Sewershed	Soil Type	K (cm/hr)	$T = \frac{\text{Depth of Water}}{K}$
McFarlane-Pine Glen	Sand to silty sand	3	1.2 - 1.75 hrs
Heart's Desire	Sandy silt tills	1	3.5 - 5.25 hrs

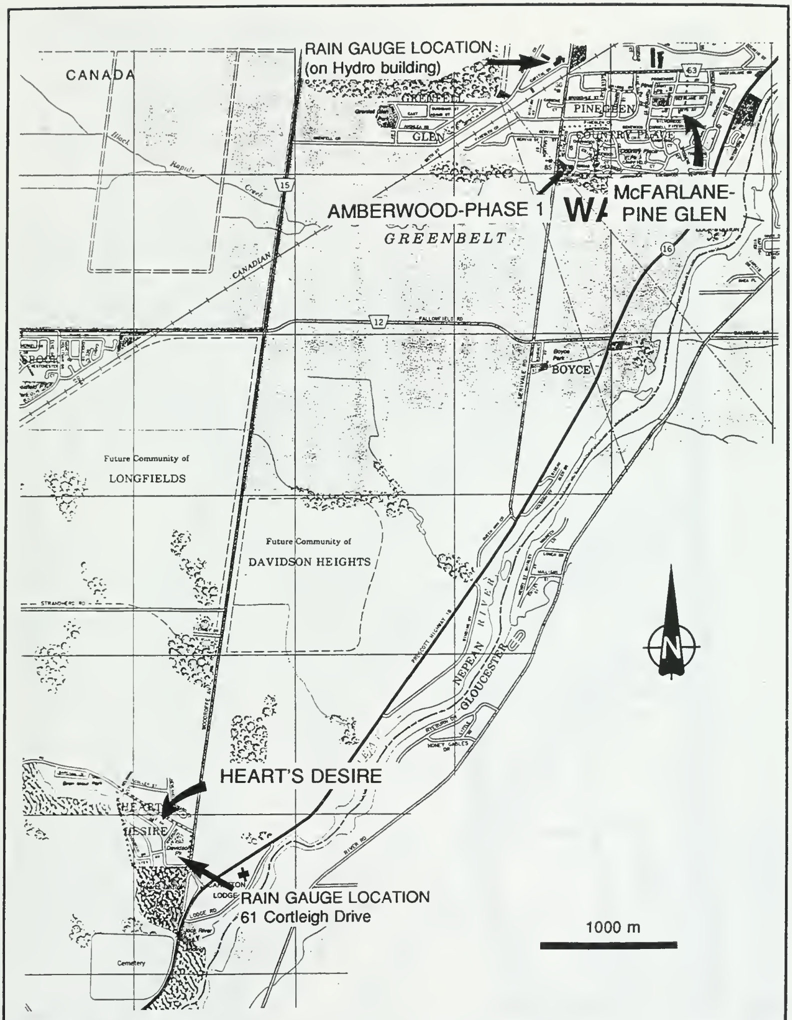

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FIGURE 40. LOCATION OF THE PERFORATED AND CONVENTIONAL SYSTEMS MONITORED BY PWA

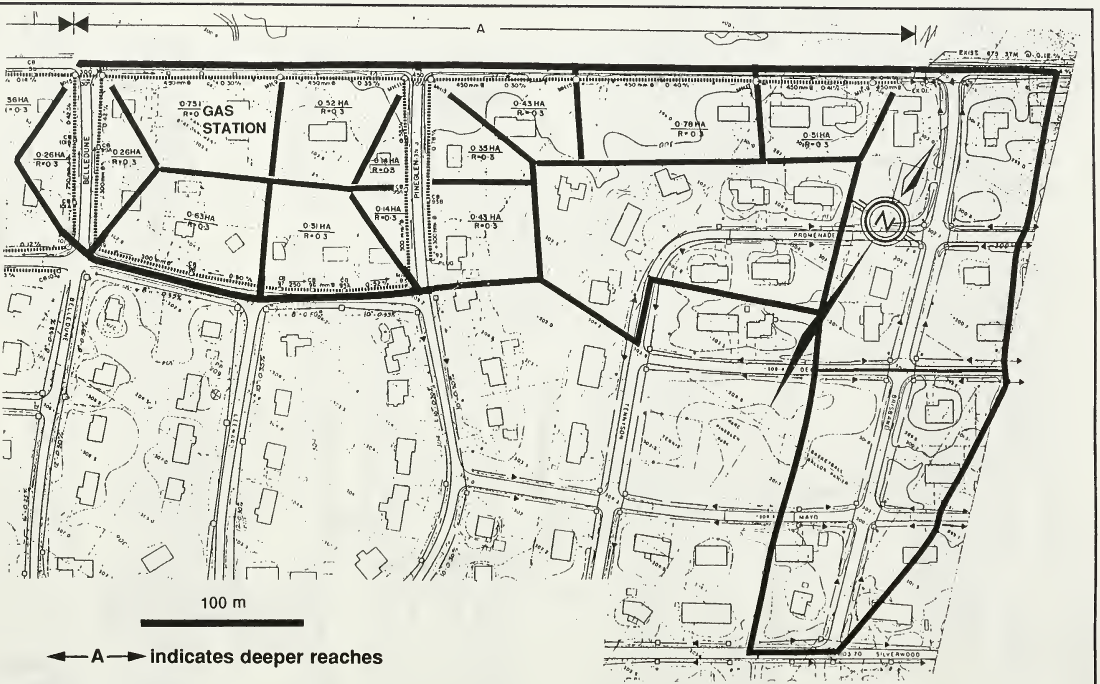

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FIGURE 41a. PLAN OF THE McFARLANE-PINE GLEN NPPS SYSTEM

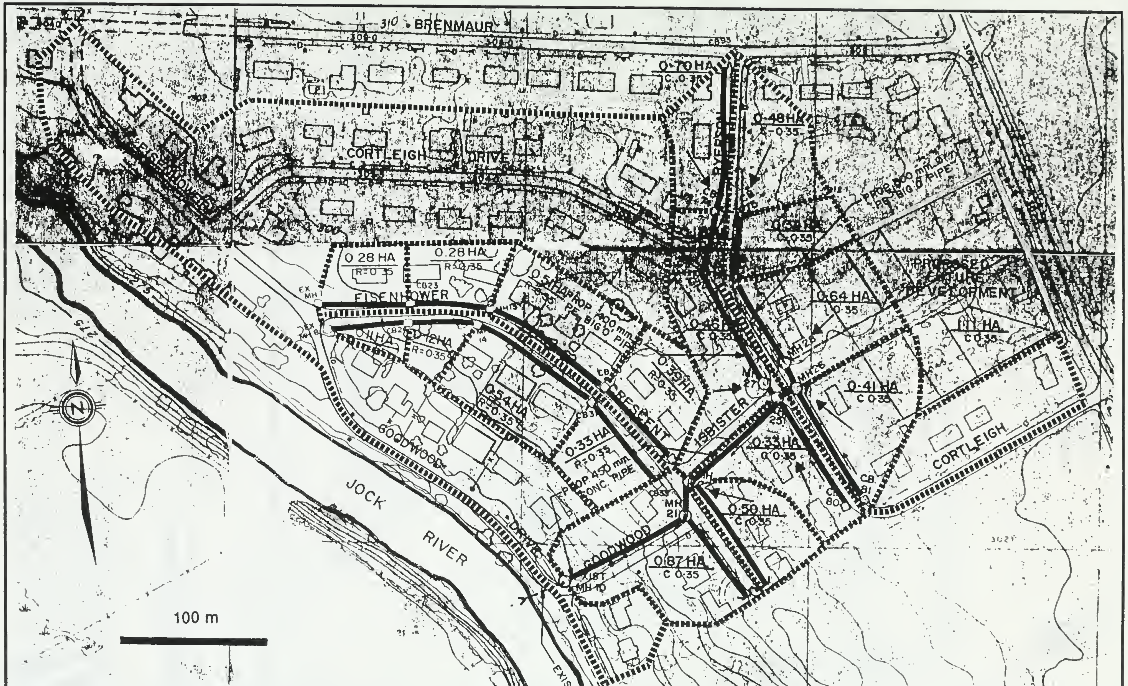

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FIGURE 41b PLAN OF THE HEART'S DESIRE NPPS SEWERSHED

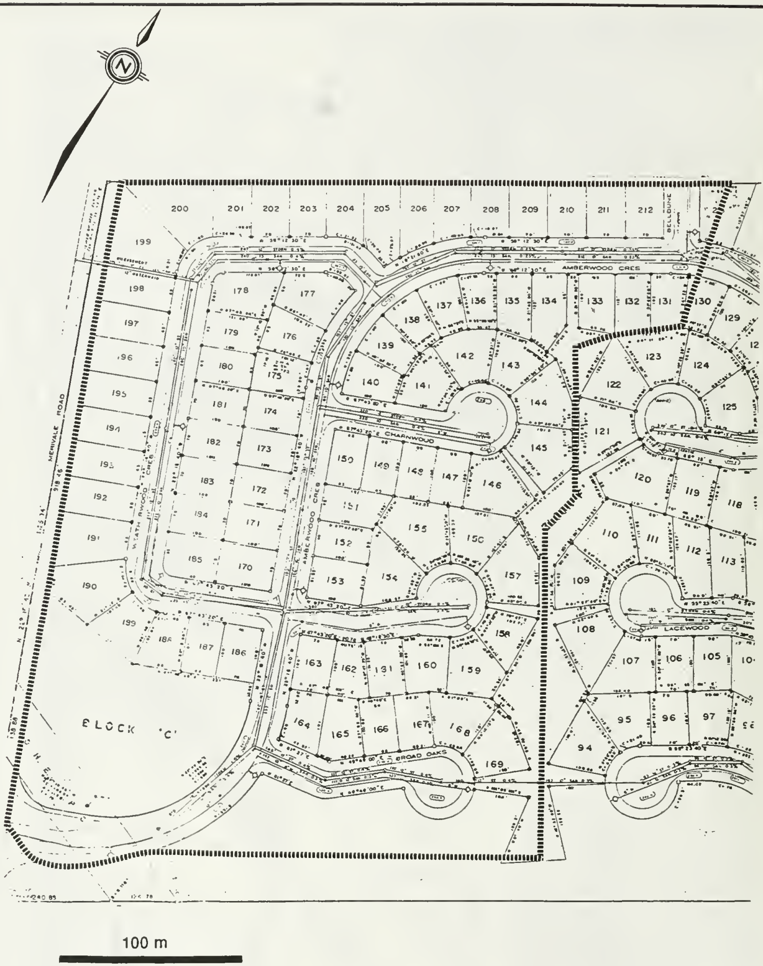

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FIGURE 41c PLAN OF THE AMBERWOOD PHASE 1 CONVENTIONAL SYSTEM SEWERSHED

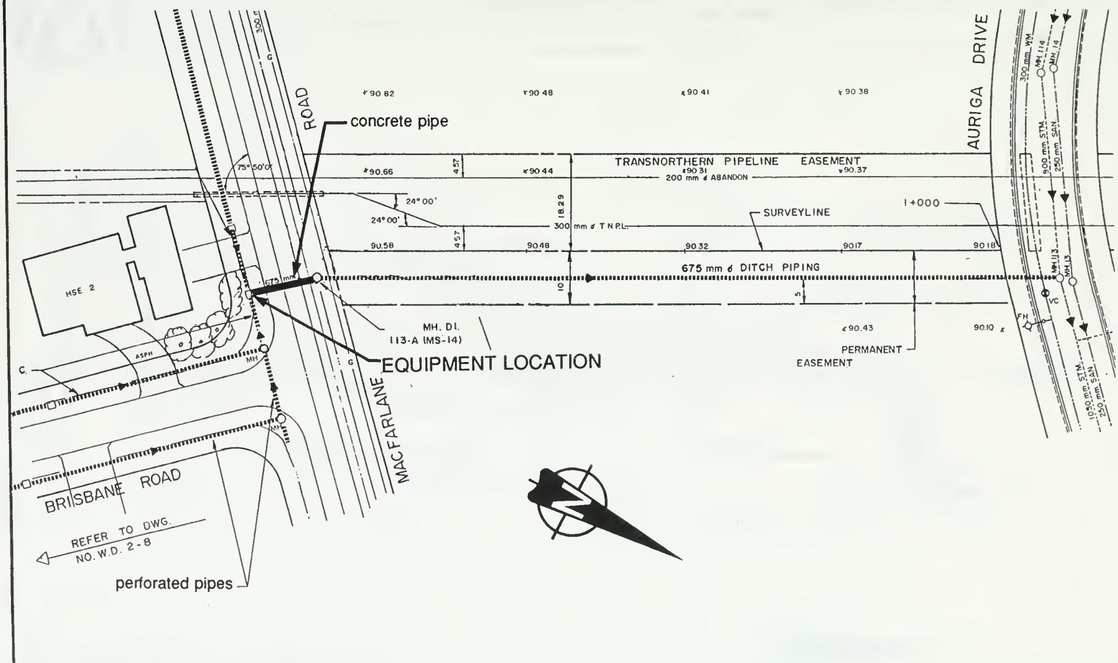

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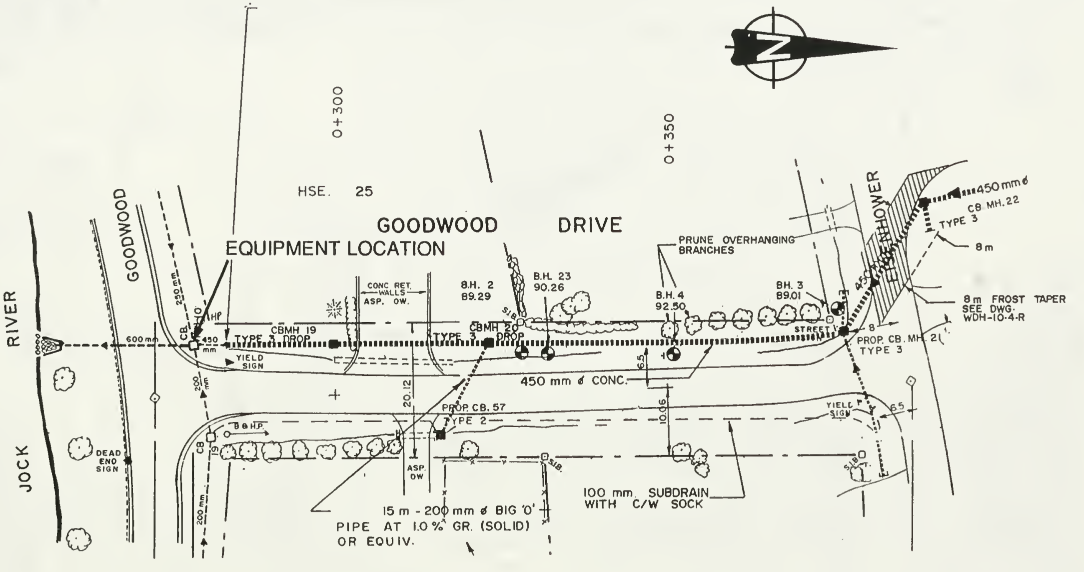
FIGURE 42a. LOCATION OF FLOW MONITORING AND WATER QUALITY SAMPLING EQUIPMENT FOR THE McFARLANE-PINE GLEN NPPS



PAUL WISNER & ASSOCIATES INC. Water Resources and Environmental Consultants Ottawa · Toronto	FIGURE 42b. LOCATION OF FLOW MONITORING AND WATER QUALITY SAMPLING EQUIPMENT FOR THE HEART'S DESIRE NPPS

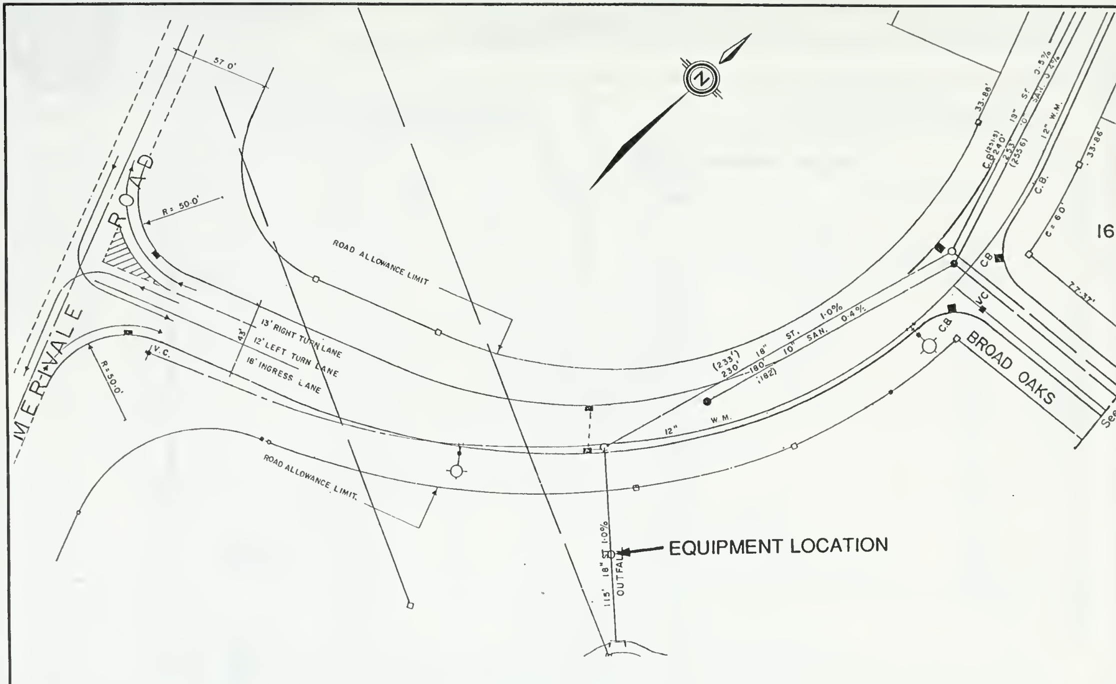


FIGURE 42c. LOCATION OF FLOW MONITORING AND WATER QUALITY SAMPLING EQUIPMENT FOR THE AMBERWOOD-PHASE 1 CONVENTIONAL SYSTEM

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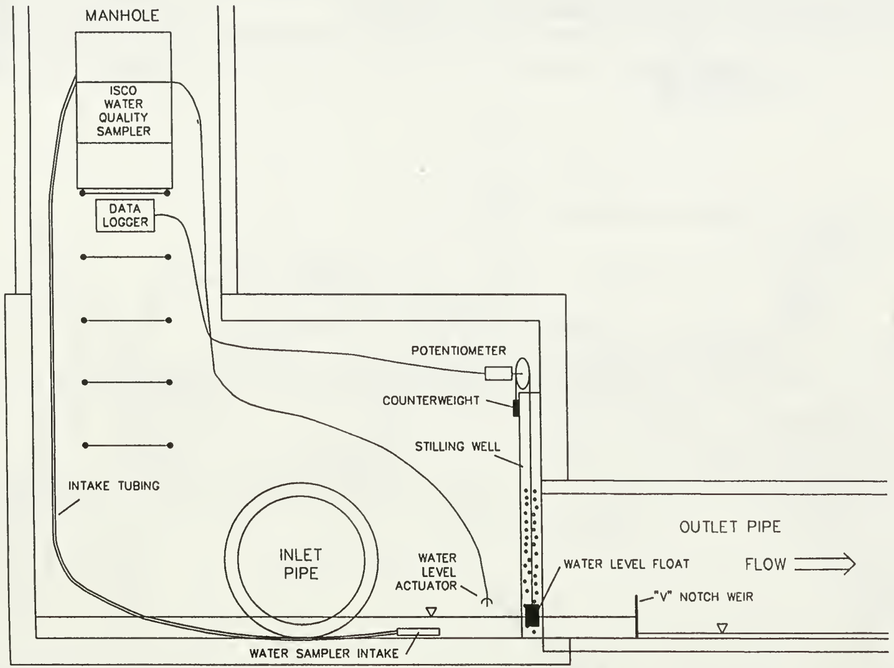

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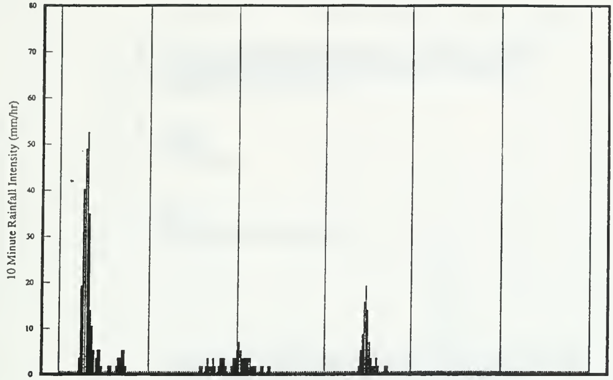
FIGURE 43. SCHEMATIC OF TYPICAL FLOW MONITORING AND WATER QUALITY SAMPLING INSTALLATION

PINE GLEN RAINFALL DATA

June 19-20 Storm 37.1 mm



McFARLANE - PINE GLEN 10.02ha

Water Quality Data - June 19-20 Storm

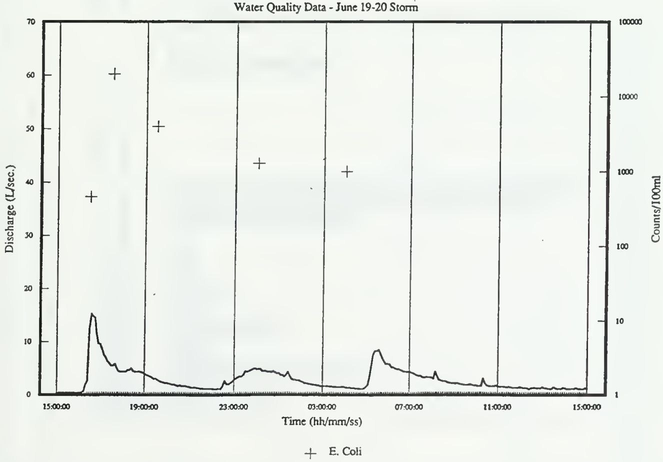


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FIGURE 44. TYPICAL HYETOGRAPHS, HYDROGRAPHS AND POLLUTOGRAPHS

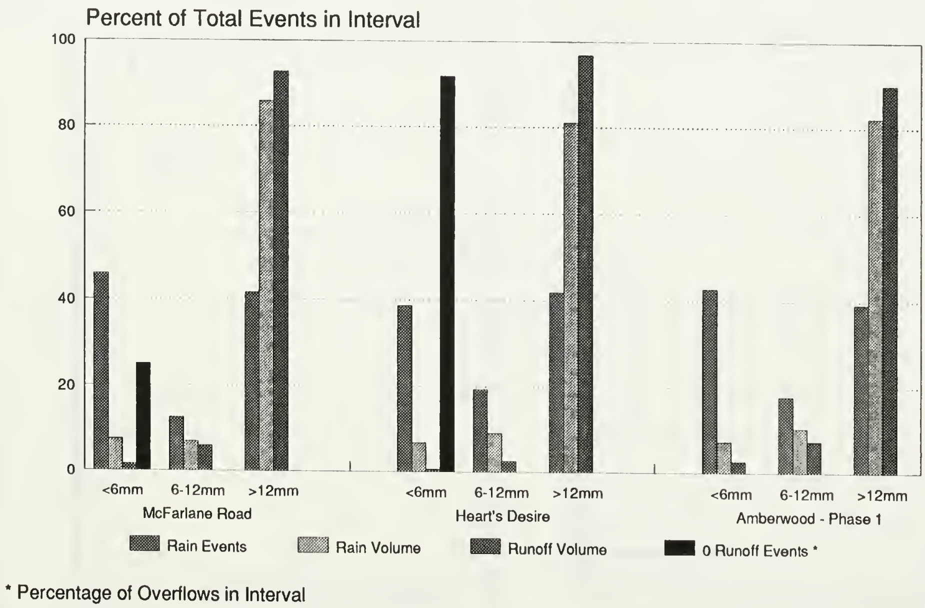

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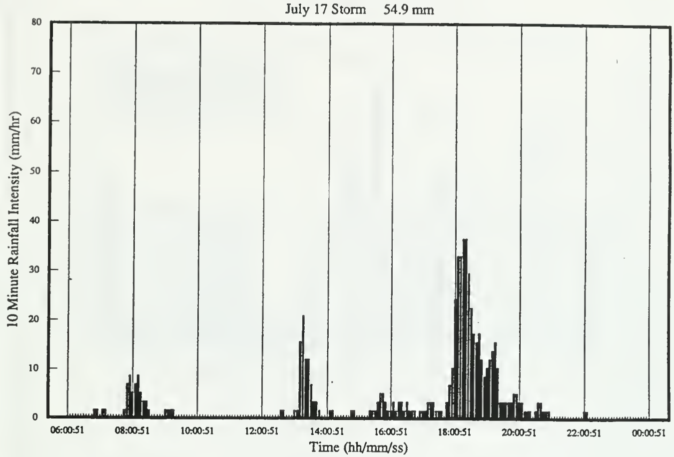
FIGURE 45 DISTRIBUTION OF NUMBER OF RAIN EVENTS, RAIN AND RUNOFF VOLUMES, AND ZERO RUNOFF EVENTS BY RAIN INTERVAL FOR McFARLANE-PINE GLEN, HEART'S DESIRE AND AMBERWOOD PHASE 1

PINE GLEN RAINFALL DATA



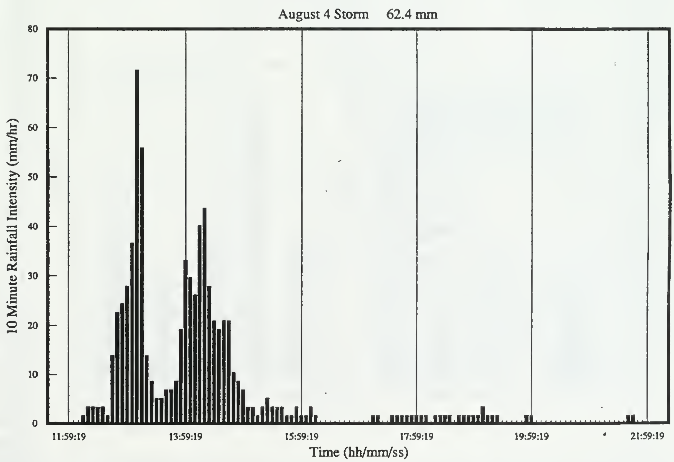


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FIGURE 46. HYETOGRAPHS FOR THE JULY 17TH AND AUGUST 4TH STORMS MEASURED AT PINE GLEN

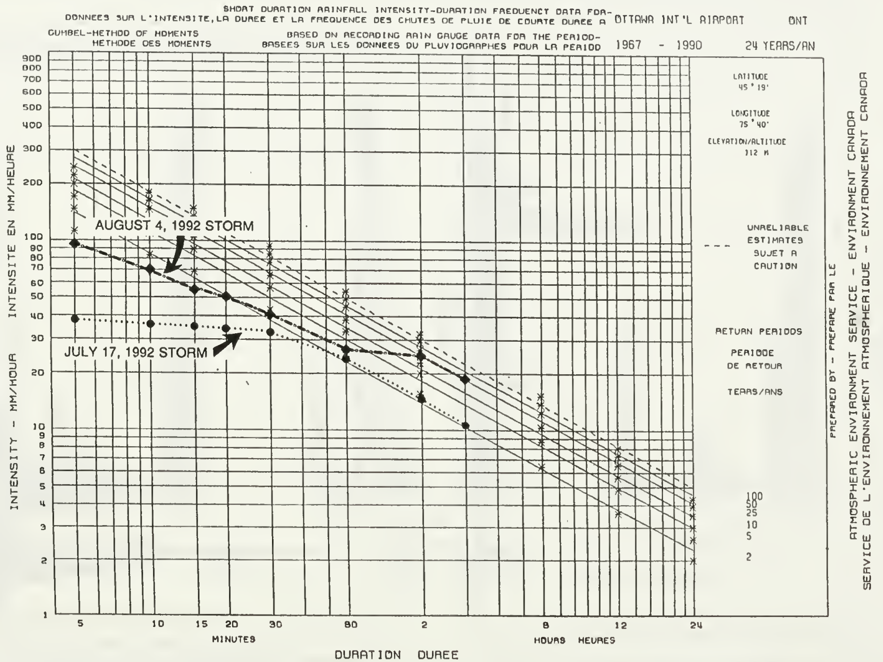

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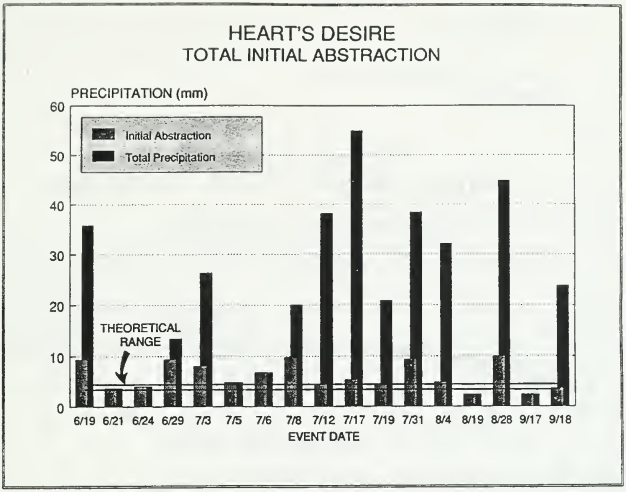
FIGURE 47 COMPARISON OF JULY 17 AND AUGUST 4 STORMS MEASURED AT PINE GLEN WITH IDF CURVES FOR THE OTTAWA INTERNATIONAL AIRPORT



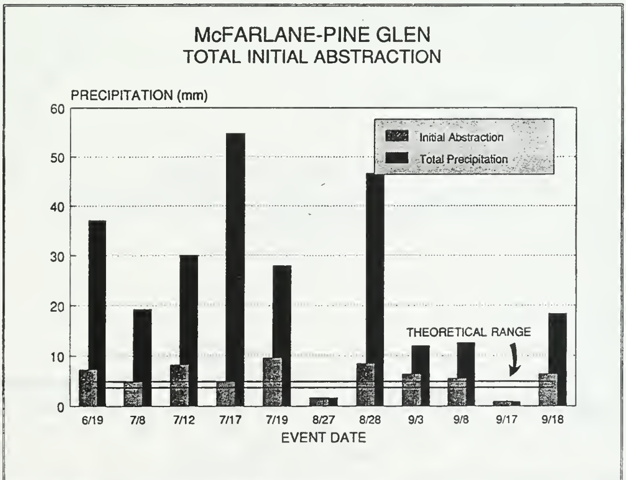


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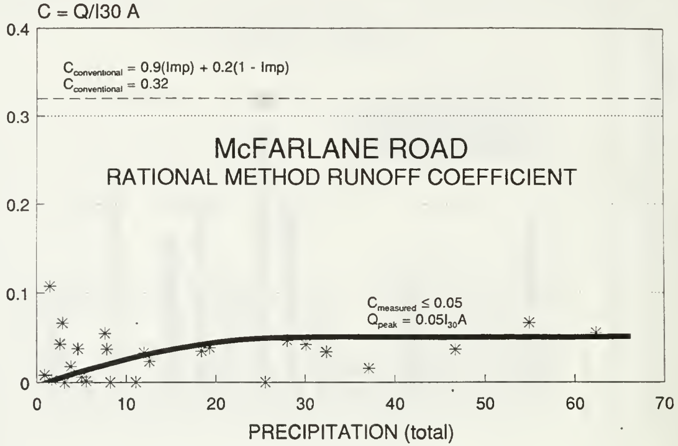
FIGURE 48 MEASURED AND THEORETICAL TOTAL INITIAL ABSTRACTION (TIA) FOR THE NPPS SITES



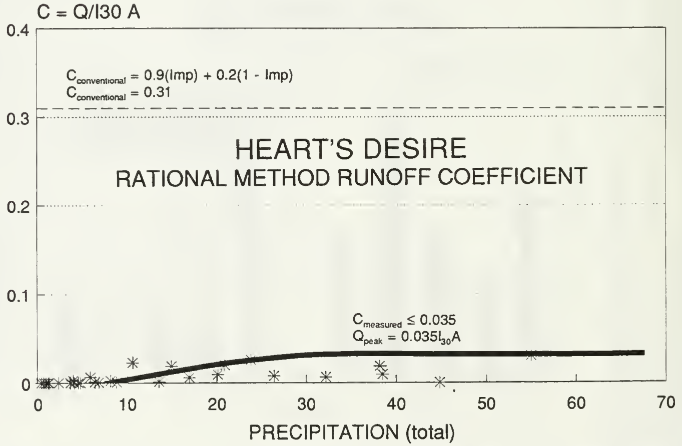


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FIGURE 49a DETERMINATION OF C USING SIMPLIFIED RATIONAL METHOD FOR McFARLANE-PINE GLEN AND HEART'S DESIRE

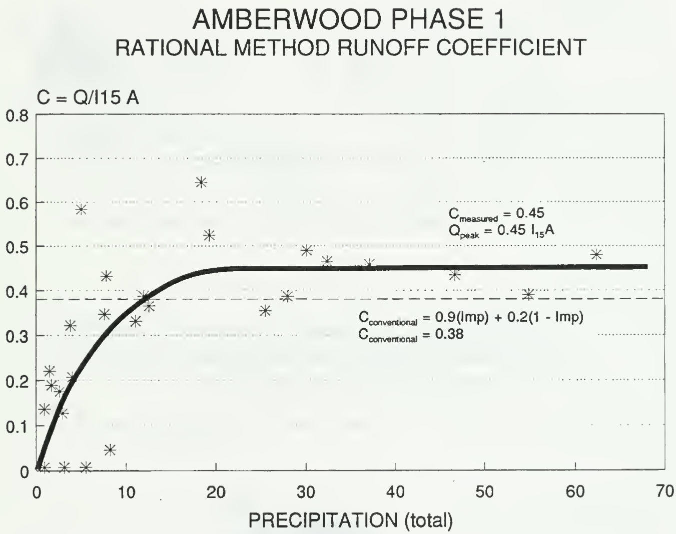

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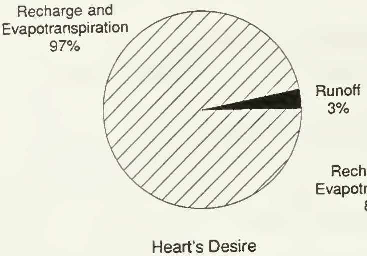
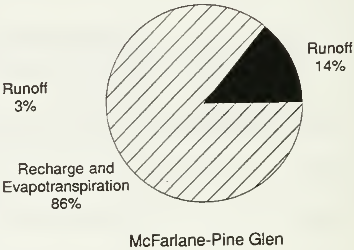
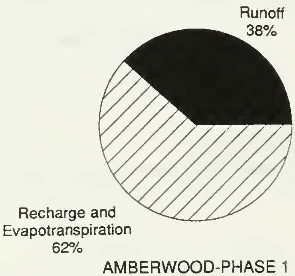
FIGURE 49b DETERMINATION OF C USING SIMPLIFIED RATIONAL METHOD FOR AMBERWOOD PHASE 1

NEPEAN PERFORATED PIPE SYSTEM
SEASONAL RUNOFF AND RECHARGE CAPABILITY

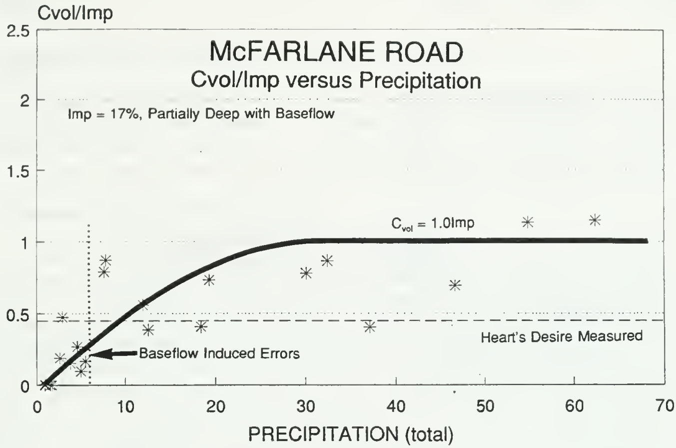




TRADITIONAL CONCRETE PIPE SYSTEM
SEASONAL RUNOFF AND RECHARGE CAPABILITY



PAUL WISNER & ASSOCIATES INC. Water Resources and Environmental Consultants Ottawa · Toronto	FIGURE 50. SEASONAL RUNOFF AND RECHARGE CAPABILITY EXPRESSED AS A PERCENTAGE OF TOTAL RAINFALL



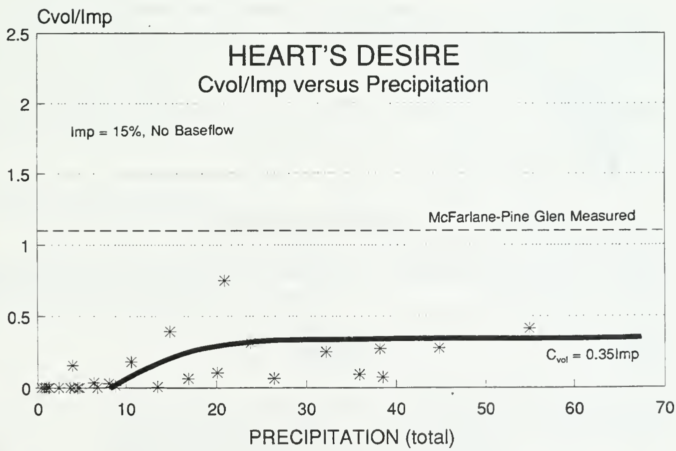


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FIGURE 51a PLOT OF C_{vol}/IMP VERSUS P FOR McFARLANE-PINE GLEN AND HEART'S DESIRE

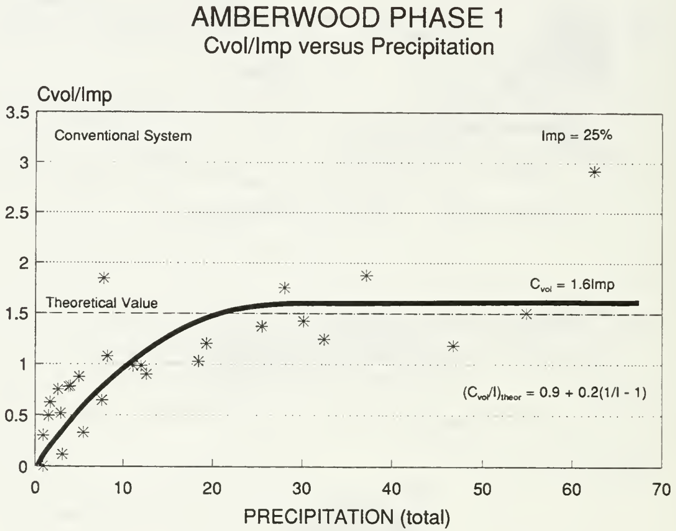

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FIGURE 51b PLOT OF C_{vol}/IMP VERSUS P FOR AMBERWOOD PHASE 1

10.0 STORMWATER AND GROUNDWATER QUALITY

Section 10.1 is devoted to the analysis of stormwater quality for both perforated pipe systems and conventional storm sewers. In Section 10.2 results of the groundwater quality monitoring program are presented and analyzed.

10.1 Stormwater Quality Monitoring

In this analysis, an attempt was made to evaluate the performance of perforated pipes in terms of stormwater quality through a comparison of data collected on perforated pipes and conventional separate storm sewer systems. Measurements of runoff volumes and pollutant concentrations at McFarlane-Pine Glen sewershed, which is drained by perforated pipes, were compared with those at Amberwood Phase I sewershed, characterized by a conventional separate sewer system. The runoff and coliform data at Heart's Desire subdivision (also drained by perforated pipes) were also incorporated in the analysis. Compared to McFarlane, this system has the advantage, of lower groundwater level and negligible baseflow. The comparison was made in terms of pollutant concentrations and pollutant loadings during the monitoring period. Pollutant concentrations in the perforated pipes were also compared with those reported in the literature.

It is reminded (as indicated in Section 9) that measurements taken at the Heart's Desire subdivision were funded through a separate contract with the City of Nepean for work requested by MOE.

Location of the automatic ISCO samples and flow monitors are described in Section 9 which also gives results of runoff measurements.

10.1.1 Pollutant Concentration

A series of pollutant concentrations at different time periods covering the course of the monitored storms were obtained. A weighted average concentration of a pollutant in a storm event was calculated as follows:

$$C_{avg} = \frac{\Sigma\, Q_i\, C_i\, t_i}{\Sigma\, Q_i\, t_i} \qquad \textbf{(19)}$$

where C_{avg} = average concentration of the storm event (M/L^3)
i = i^{th} sampling period = 1,2,3,...,n, (n= number of sampling periods)
C_i = pollutant concentration in the i^{th} period (M/L^3)
Q_i = runoff flow rate during the ith period (L^3/T)
t_i = time span of the i^{th} period (T)

Both quantity and quality of stormwater runoff were monitored during the period from June 15 to September 20, 1992. The average pollutant concentration of all storm events for the complete monitoring period at a given site was calculated using the following equation:

$$C = \frac{\Sigma V_j \, C_{avg,\,j}}{\Sigma V_j} \qquad \textbf{(20)}$$

where C = overall average pollutant concentration during the monitoring period (M/L^3)

j = j^{th} storm event with sampling for concentration during the monitoring period = 1,2,3,...,m, where m is the total number of storm events that have concentration measurements.

$C_{avg,\,j}$ = average concentration of the j^{th} storm event (M/L^3)

V_j = total runoff volume of the j^{th} storm event (L^3)

Table 20 lists the number of events for which water quality samples were obtained.

Table 20: Number of Storms with Concentration Sampling

Location	Number of Storms
McFarlane	7
Amberwood	7
Heart's Desire	9

Averaged coliform concentrations for each event at the three monitoring sites are shown in Figures 52 to 54. Concentrations vary significantly from event to event. Table 21 lists the extreme values of the variation during the monitoring period. Concentrations of other constituents are listed in Tables 22 and 23. The tables show that coliforms have the greatest variability in concentration of all pollutants for all the monitoring sites. Ratios of the maximum to minimum concentrations range from 25 to 187 for coliforms, but only from 1.3 to 20 for other constituents.

Table 21: Observed Maximum and Minimum Event Coliform Concentrations (counts/100 mL)

Location	Type	Total Coliform	Fecal Coliform	E. Coliform
McFarlane	Maximum	15,552	8415	6149
	Minimum	98	45	45
	Ratio (Max/Min)	159	187	136
Amberwood	Maximum	9851	1366	2102
	Minimum	390	74	34
	Ratio (Max/Min)	25	18	62
Heart's Desire	Maximum	-	-	11,241
	Minimum	-	-	60
	Ratio (Max/Min)	-	-	187

The pattern of pollutographs is also somewhat different for coliforms. Because of these differences between the variability of concentrations of coliforms and other constituents, the analysis of pollution characteristics of the two groups of constituents are conducted separately in the following sections.

Table 22: Event Pollutant Concentrations at McFarlane (mg/L)

Date of Event	Pollutants							
	TSS	Nitrate	Phosphate	TKN	Copper	Lead	Zinc	Chloride
June 19, 1992	100	1.72	0.41	0.75	0.017	0.0001	0.052	310
July 8, 1992	125	2.04	0.74	0.45	0.014	0.002	0.056	205
July 17, 1992	7	1.66	0.35	0.65	0.000	0.0001	0.038	98
August 8, 1992	10	4.18	0.41	0.28	0.000	0.0001	0.034	248
Sept. 3, 1992	46	2.38	0.13	0.63	-	-	-	-
Maximum	125	2.38	0.74	0.75	0.017	0.002	0.056	310
Minimum	7	1.66	0.13	0.28	0.000	0.0001	0.034	98
Ratio (Max/Min)	18	1.4	5.7	2.7	-	20	1.6	1.3

Table 23: Event Pollutant Concentrations at Amberwood (mg/L)

Date of Event	Pollutants							
	TSS	Nitrate	Phosphate	TKN	Copper	Lead	Zinc	Chloride
June 19, 1992	271	0.64	0.62	0.53	0.009	0.00	0.072	17
June 24, 1992	39	2.56	0.52	1.52	0.025	0.002	0.073	63
June 29, 1992	47	1.46	0.27	0.97	0.01	0.002	0.049	16
July 3, 1992	19	2.04	0.19	0.92	0.00	0.004	0.064	4
July 8, 1992	37	1.02	0.50	0.69	0.005	0.005	0.035	5
Sept. 3, 1992	14	0.86	-	-	-	-	-	-
Maximum	271	2.56	0.62	1.52	0.025	0.005	0.073	63
Minimum	14	0.64	0.19	0.53	0.00	0.00	0.035	4
Ratio (Max/Min)	19	4	3.3	2.9	-	-	2.1	15.8

10.1.1.1 Coliform Concentration

The distribution of coliform concentrations in the three systems (McFarlane, Heart's Desire and Amberwood) are used to assess the impacts from various sites.

The number of dry days prior to the rainfall as well as the total precipitation of each event are included in Figures 52 to 54 to examine the hydrological response of coliform concentration in the runoff. High coliform concentrations (especially total coliforms), were observed in the event of June 19 at all three sites. Although the longest dry period is associated with the June 19 event at McFarlane and Amberwood, it is not the case at Heart's Desire. The event of July 8 has a shorter dry period, but higher coliform concentrations than the event of August 8 at McFarlane. *A correlation between the concentration and dry period does not seem to exist.*

The comparison between the coliform concentration and the amount of total precipitation also revealed that the two are not necessarily related. As an example, rainfall was more intense on June 19 than August 8 (see Figure 52), but coliforms were higher in concentration in the former event. The effects of the first flush are not obvious in the measured data for coliforms.

Average concentrations of all pollutants of each monitoring site during the monitoring period were calculated using Equation 20. Two scenarios were examined in the calculations: inclusion and exclusion of the base flow loading. In the first case, pollutant loadings during a storm event were obtained by multiplying the averaged event concentrations by the total runoff generated by the storm. In the second method, the base flow loading (represented by the measurement of dry weather flow) was subtracted from the total loading. The sum of the loadings of each event was then divided by the corresponding total runoff volume to give the overall average concentration at a monitoring site.

Results with and without baseflow loadings are illustrated in Figures 55 and 56 respectively. The exclusion of the base flow loading leads to lower overall average concentrations.

The figures show that for several events the average coliform concentrations per event are somewhat higher in perforated pipes (McFarlane and Heart's Desire) than in conventional pipes (Amberwood).

In order to conduct a further evaluation of the variability of coliform concentration, the geometric mean and standard deviation for all the individual samples over the monitoring period were also calculated. The results are shown in Table 24. The Table also shows that coliform concentrations are somewhat higher in perforated pipes than in conventional pipes.

Table 24: Geometric Mean and Standard Deviation of Coliform Concentrations (counts/100 mL)

Location	Constituent	Geometric Mean	Standard Deviation
McFarlane	Total Coliform	970	24,772
	Fecal Coliform	368	5,140
	E. Coli.	477	2,971
Heart's Desire	Total Coliform	---	---
	Fecal Coliform	---	---
	E. Coli.	551	3,676
Amberwood	Total Coliform	1,198	4,941
	Fecal Coliform	319	1,227
	E. Coli.	288	1,672

The very high, standard deviation values show that due to the limited number of events for which pollutant concentrations are measured, a direct comparison of the two systems based on concentrations may not be conclusive.

Another comparison was made using log E.Coli samples for each subwatershed. Results are shown in Figures 57 to 59. The comparison in Figure 60, shows that the fitted frequency curve and the 90% confidence interval for the three systems vary with the percentage of results less than a given concentration. Within a certain range the differences are less than the 90% confidence interval. (i.e. there is no significant difference at the 90% confidence level).

Assuming, however, that the average concentrations is indeed greater for the perforated pipes, the pollution impact has to consider the loading given by the combined effect of mean concentration and runoff volume. Figure 61 compares the product of E.Coli concentration and flow. It is seen that even for higher concentrations in the perforated pipes the impact on receiving waters or the loading of a treatment pond is smaller than for the traditional system.

Other interpretations such as Gamma distributions used in NURP studies in the USA are possible, but the need for a greater number of samples in order to derive a more rigorous comparison is obvious.

10.1.1.2 Other Pollutants

Table 25 summarizes the overall average pollutant concentrations, obtained using Equation 20, for all parameters monitored at each site. The table shows that chloride concentrations are significantly higher at McFarlane than Amberwood; the difference is more than 8-fold. Considering that runoff in perforated pipes at McFarlane is subject to exchanges with groundwater due to the deep layout of some pipes, the higher concentration of chloride may be caused by the release of salt used for de-icing that had seeped into the ground.

A comparison of the range of pollutant concentrations was also made between the present study and the results of three study cases reported in the literature as shown in Table 26. It was found that the pollutant concentrations observed in the present study have about the same range as reported in the other studies.

Average concentration ratios for the rest of the constituents are shown in Table 27. The table shows that, except for nitrate, pollutant concentrations are somewhat lower in perforated pipe systems (McFarlane) than in conventional systems (Amberwood). High nitrate concentrations in McFarlane are mainly explained by the interference of groundwater, where high concentrations of nitrate, originating from septic tanks, were observed.

Table 25: Weighted Average Pollutant Concentrations in Stormwater Runoff

Parameter		This study (from 19/6 to 18/9/1992) McFarlane	Amberwood	Heart's Desire	Windsor Annual Mean (Droste & Hartt, 1975)	Sault Ste. Marie (Marsalek, 1990)
Tot Coli (counts/100 ml)		3190	4920		64000	
F. Coli (counts/100 ml)		1656	724		8200	
E. Coli (counts/100 ml)		1250	706	2778		
Cu	(mg/L)	0.0047	0.0063			0.041
Pb	(mg/L)	0.00037	0.002			0.16
Zn	(mg/L)	0.042	0.062			0.263
TSS	(mg/L)	39	138		279	
NO3-N	(mg/L)	1.93	1.1		1.16	
PO4	(mg/L)	0.4	0.46		0.44	0.246
Cl	(mg/L)	157	12		72	
DO	(mg/L)	11.3	11.1			
TKN	(mg/L)	0.61	0.7			
NH4-N	(mg/L)	0	0.19		0.054	0.58
BOD	(mg/L)	2.6	3.2		20.5	

Literature cited:

1. Droste, R.L. and Hartt, J.P. 1975. "Quality and variation of pollutant loads in urban stormwater runoff'. Can. J. Civ. Eng. Vol 2, No.4, pp. 418-429.
2. Marsalek, J. 1990. "Evaluation of pollutant loads from urban nonpiont sources." Wat. Sci. Tech. Vol. 22, No. 10/11, pp. 23-30.

Table 26: Range of Pollutant Concentrations in Stormwater Runoff

Parameter		This study		Windsor Annual Mean (Droste & Hartt, 1975)	Sault Ste. Marie Ontario (Marsalek, 1990)	Fisher Glen, Nepean (PWA, 1991)
		McFarLane	Amberwood			
Tot Coli (counts/100 ml)		20-125,000	0-20,000	200-1,200,000		
F. Coli (counts/100 ml)		20-26,000	0-2100	10-200,000		0-100,000
E. Coli (counts/100 ml)		20-20,000	0-1500			10-100,000
Cu	(mg/L)	0-0.02	0-0.08		0.032-0.052	
Pb	(mg/L)	0-0.005	0-0.01		0.086-0.282	
Zn	(mg/L)	0.02-0.2	0.01-0.35		0.219-0.316	
TSS	(mg/L)	0-1720	0-1250	23-1230		2-668
NO3-N	(mg/L)	0.92-5	0.24-5.98	0-4.7		
PO4	(mg/L)	0.11-0.99	0.11-1.13	0-2.5	0.191-0.318	
Cl	(mg/L)	78-454	0-174	4-1585		
DO	(mg/L)	8.6-11.7	9.9-12.2			
TKN	(mg/L)	0.17-1.76	0-3.21			

Literature cited:

1. Droste, R.L. and Hartt, J.P. 1975. "Quality and variavtion of pollutant loads in urban stormwater runoff'. Can. J. Civ. Eng. Vol 2, No.4, pp. 418-429.
2. Marsalek, J. 1990. "Evaluation of pollutant loads from urban nonplont sources." Wat. Sci. Tech. Vol. 22, No. 10/11, pp. 23-30.
3. Paul Wisner & Associates Inc. 1991. "City of Nepean stormwater pond automation study." Raw water quality data and plots.

Table 27: Ratio of Flow Weighted Average Pollutant Concentrations of McFarlane to Amberwood (P.P. = Perforated Pipes, C.P. = Conventional Concrete Pipes)

Constituent	Average Ratio ($C_{P.P.}/C_{C.P.}$)
TSS	0.28
NO_3	1.75
PO_4	0.87
TKN	0.87
BOD_5	0.80
Cu	0.75
Pb	0.19
Zn	0.67

While perforated pipe systems (swales to pipes) seem to be able to reduce the amount of suspended solids in runoff, more sampling is required before a conclusion can be reached.

10.1.2 Pollutant Loading

Total pollutant loadings during the monitoring period from June 15 to September 20, 1992 were also calculated. The loadings were obtained by multiplying overall average concentrations by the total runoff volumes generated during this period.

Two values of loading rate were respectively calculated for each pollutant (with and without base flow loading). Values of total loading rates are tabulated in Table 28. For both scenarios, all the constituents, except chloride, have lower loading rates in perforated pipes than in conventional storm sewers. The TSS loading is about 10 times lower. This is mainly attributed to the significant reduction in stormwater runoff volumes in perforated pipes systems.

As stated previously, part of the perforated pipe system at McFarlane is deeper than a normal perforated pipe system. As a result, the exchange between runoff and groundwater leads to high concentrations of chloride, which originate from de-icing on streets. Therefore, the chloride loading rate, as shown in Table 28, is higher at McFarlane than at Amberwood. In addition to the high concentration of chloride, another effect of the deep layout of perforated pipes at McFarlane is that it generally yields higher stormwater runoff than in Heart's Desire.

Table 28: Pollutant Loadings by Perforated and Conventional Pipes During the Period from June 15 to Sept. 20, 1992

Parameter	Unit	Base flow INCLUDED		Base flow EXCLUDED	
		Perforated Pipe	Conventional Pipe	Perforated Pipe	Conventional Pipe
		McFarlane	Amberwood	McFarlane	Amberwood
NO_3-N	kg/ha	1.18	1.90	0.26	---
TKN	kg/ha	0.37	1.19	0.33	1.09
PO_4	kg/ha	0.25	0.79	0.16	0.65
TSS	kg/ha	24	235	22	232
Cl	kg/ha	96	21	23	---
Cu	kg/ha	0.0028	0.01100	0.00270	0.01100
Pb	kg/ha	0.00023	0.00340	0.00023	0.00340
Zn	kg/ha	0.026	0.110	0.022	0.097

The effect of baseflow on the comparison of loading rates between perforated and conventional pipe systems for all the constituents is summarized in Table 29.

Table 29: Ratio of Pollutant Loadings of Perforated to Conventional Pipe Systems

Parameter	Base Flow Loading INCLUDED	Base Flow Loading EXCLUDED
NO_3-N	0.62	-
TKN	0.31	0.30
PO_4	0.32	0.25
TSS	0.10	0.10
Cl	4.63	-
Cu	0.25	0.25
Pb	0.07	0.07
Zn	0.24	0.23

10.2 Groundwater Quality Analysis

An analysis of groundwater quality data was conducted. Sampling was made on the site of Promenade Avenue, Nepean, located within a sewershed that has the outlet at McFarlane Road, where the quantity and quality of surface stormwater runoff were also monitored. The groundwater quality results were intended to be used for the evaluation of the impact of perforated pipes on groundwater quality. Monitoring data reported herein covered the period from March 6 to August 31, 1992.

Three native soils in the site have been divided into three layers: a) a fine to medium sand, b) sandy silty clay, and c) silty clay. A thin layer (100 to 200 mm) of topsoil overlays these soils layers. The uppermost soil layer is characterized by loosely compacted, brownish, fine to medium grain sand with variable amounts of silt (up to 25%). The thickness of this layer varies from 0.6 to 1.0 metres. This second soil layer is composed of a tannish/light brown sandy silty clay with thickness varying between 0.1 to 0.3 metres. This soil is also firmly compacted and may contain up to 15% very fine sand. The contacts between the first and the second soil layers are distinct (well defined). The lowermost soil layer is composed of silty blue grey clay of marine origin. This layer is also highly compacted. The contacts between the second and third soil layer are gradual and difficult to discern. Borehole logs are provided in Appendix G.

Piezometers and observation wells were installed during the construction of a new grass swale-perforated pipe system in the fall of 1991. The site which is located within the Pine Glen subdivision in Nepean is described in Section 4 where the location of the piezometers and observation wells are shown in Figures 4a and 4b.

Groundwater samples were taken at three piezometer locations denoted as P1, P2 and P3, respectively. Samples were also taken at the inlet and outlet of the perforated pipe system. The site of P2 was denoted as "upstream" since it is located upstream of the perforated pipe system with respect to the groundwater flow. Piezometer locations P1 and P3 represent the downstream section. The perforated pipe system, in which the groundwater quality samples were taken, is deeper than a typical system, implying greater effects with respect to groundwater contamination.

Two sources of groundwater contamination were identified for the study area: one is the seepage of sanitary waste from nearby septic tanks and the other is stormwater runoff collected by perforated pipes.

10.2.1 Data Analysis

Water quality constituents detected during the monitoring period included dissolved oxygen (DO), nitrate, total phosphorus, pH, chloride, biochemical oxygen demand (BOD_5), fecal and total coliforms as well as heavy metals. Grab samples were taken at each of the monitoring locations. The number of site visits during which water quality samples were taken is listed in Table 30.

Table 30: Number of Groundwater Quality Sampling

Location	No. of Site Visits where samples were taken
P1	8
P2	5
P3	4
Inflow	2
Outfall	3

Note: Because of the absence of water, some samples could not be taken

The average concentration of each constituent was obtained by averaging over the data monitoring period. Pollutant concentrations averaged over the monitoring period are summarized in Table 31 and Figures 62 to 64. Because of the limited duration of the monitoring period, general remarks can only be made.

Table 31: Average Pollutant Concentrations in Groundwater

Parameter	Unit	Upstream (P2)	Downstream (P1+P3)/2	Inflow	Outfall	P1	P3
Tot.Coli	Counts/100 mL	23	167	71	328	129	204
F.Coli	Counts/100 mL	3	3	41	213	2	3
NO3-N	mg/L	8.03	3.24	4.16	7.91	6.34	0.14
NH4-N	mg/L	0.21	0.22			0.12	0.31
TKN	mg/L	0.40	0.27			0.20	0.34
Org-N	mg/L	0.19	0.06			0.08	0.03
Tot P	mg/L	0.17	0.15	0.27	0.03	0.17	0.12
BOD5	mg/L	1.0	11.8			2.0	21.5
DO	mg/L	8.2	5.8			6.8	4.8
Chloride	mg/L	63	213	99	180	204	222
pH		7.83	7.6	6.41	7.91	7.85	7.38
Zn	mg/L	0	0	0.05		0	0
Cu	mg/L	0	0	0		0	0
Pb	mg/L	0	0	0		0	0

The analysis was carried out for each constituent as described in the following:

Dissolved Oxygen: Figure 62 shows that dissolved oxygen (DO) in groundwater is in the range of 6 to 8 mg/L, which is above the minimum acceptable level of 4 mg/L. This suggests that groundwater is in an aerobic environment, the condition at which nitrification (conversion of ammonia nitrogen to nitrate nitrogen in the presence of microorganisms), can take place.

Phosphorus: Phosphorus concentration in an uncontaminated water body is rarely high (in the order of 0.01 - 0.02 mg/L, McNeely *et al.*, 1979 and CCME, 1992). Field measurements revealed that phosphorus concentrations in the area of interest are far higher than the background concentration in uncontaminated water (see Figure 64), which indicates that the groundwater system in the study area has been contaminated by sanitary wastes. Human excrement and household detergents contribute an appreciable amount of phosphorus. Through percolation from septic tanks, phosphorus can reach the groundwater table, resulting in high phosphorus concentrations.

Figure 64 shows that the concentration of total phosphorus upstream (with respect to the direction of groundwater flow) of the perforated pipe system is 0.18 mg/L. Such an elevated phosphorus concentration must be attributed to the discharge by septic tanks since perforated pipes are located further downstream.

Nitrogen: Groundwater is under aerobic conditions in the study area, which results in nitrification. As a result, high levels of nitrate nitrogen, which are converted from the wastes discharged by septic tanks, are observed.

From Figures 62 and 64, it is seen that nitrogen concentrations upstream are consistently higher than those downstream: nitrogen is more than twice as much; organic nitrogen decreased about 3 folds from upstream to downstream; and Total Kjeldahl Nitrogen (TKN) is also higher upstream. The only exception is that ammonia nitrogen at the two locations is at the same level, which might be due to the supplement of ammonia nitrogen from the conversion of organic nitrogen as contaminants travel downstream in the groundwater.

There are two possibilities that may lead to the decrease in nitrogen concentrations from upstream to downstream. One is the biochemical degradation that accompanies the transport process of pollutants; the other is the dilution by stormwater runoff collected by perforated pipes which are located between the two monitoring sites (upstream and downstream). One or both of these factors must have been the cause of the distribution pattern of nitrogen.

The above observations indicate that groundwater contamination is more critical in areas affected by septic tanks (upstream) than in those associated with perforated pipes (downstream).

BOD_5: A BOD concentration of 21.5 mg/L was detected at P3 (see Table 31). For a Median urban site, BOD concentration in stormwater runoff is reported to be less than 13 mg/L (Novotny, 1992). In the present study, the BOD concentration in stormwater runoff was found to be 2.6 mg/L at McFarlane and 3.2 mg/L at Amberwood (see Section 10.1). This implies an additional pollutant source other than the runoff collected by the perforated pipes, namely septic tanks.

Heavy Metals: Table 31 shows that the presence of Zn, Cu and Pb in the groundwater is nil. This suggests that the groundwater is mainly subject to contamination by organics and nutrients, which are the major components of sanitary wastes. It once again confirms that septic tank is the major source of pollutants.

Chloride: Chloride concentration at the upstream station, which is the closest to septic tanks, is the lowest of all sampling sites as illustrated in Table 31. This suggests that septic tanks are not responsible for chloride concentrations. On the other hand, the low concentration of chloride in the catchment inflow also excludes the possibility that stormwater runoff is the major source of chloride in both downstream groundwater and at the outlet of the perforated pipe system.

Compared to the groundwater at the downstream location, the outflow has a lower concentration of chloride, which might be the result of exchange and mixing of water between groundwater and catchment inflow since the concentration of chloride is lower in the inflow than in the groundwater.

Coliform: Coliforms are indicators of fecal contamination of water and therefore possible presence of intestinal parasites or pathogens. The results of coliform tests show that the concentration of fecal coliforms in groundwater (at both upstream and downstream) are very low. Total coliforms upstream are also detected to be low in concentration. This indicates that bacteria from nearby septic tanks do not reach the groundwater, which might be attributable to the effects of filtration by pore media.

pH: The groundwater quality is in normal condition as far as the pH value is concerned. Although the inflow has a low value of pH, elevated pH's were observed in the groundwater system. This might be attributed to the natural buffering capacity of the groundwater system provided by the carbonate equilibria between H_2CO_3 and HCO_3^-.

It is reminded that these conclusions are based on limited sampling on a site with specific conditions (high groundwater and effect of septic tanks). The main conclusion is that perforated pipe systems should be above groundwater levels. Studies by E. Graham indicate that a minimum layer of 0.6 m between the bottom of the storage and the groundwater level is recommended. As for other infiltration BMP's, hydrogeological investigations prior to implementation are recommended.

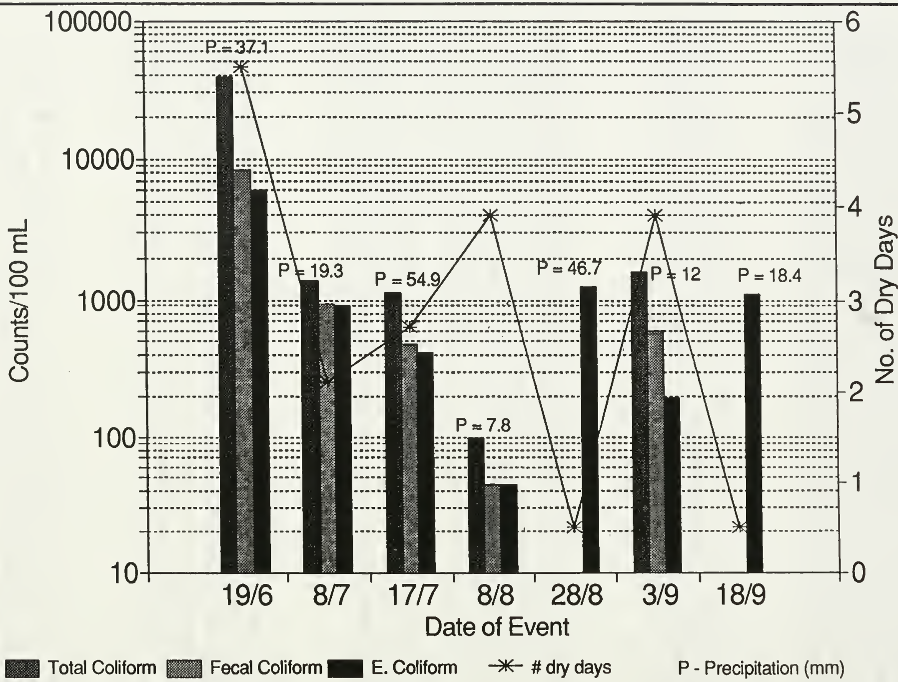

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FIGURE 52. AVERAGED COLIFORM CONCENTRATIONS - MACFARLANE

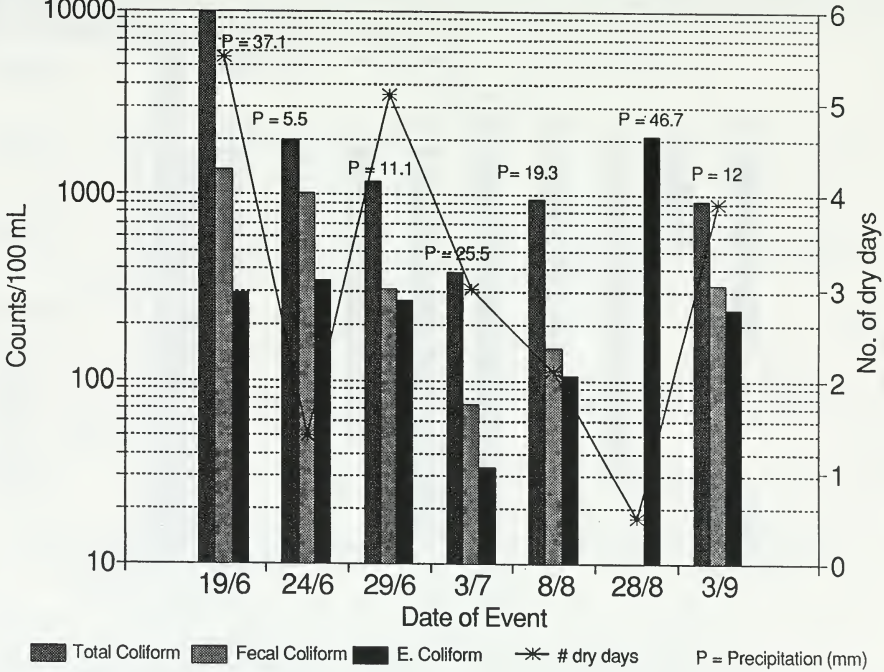

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FIGURE 53. AVERAGED COLIFORM CONCENTRATIONS - AMBERWOOD

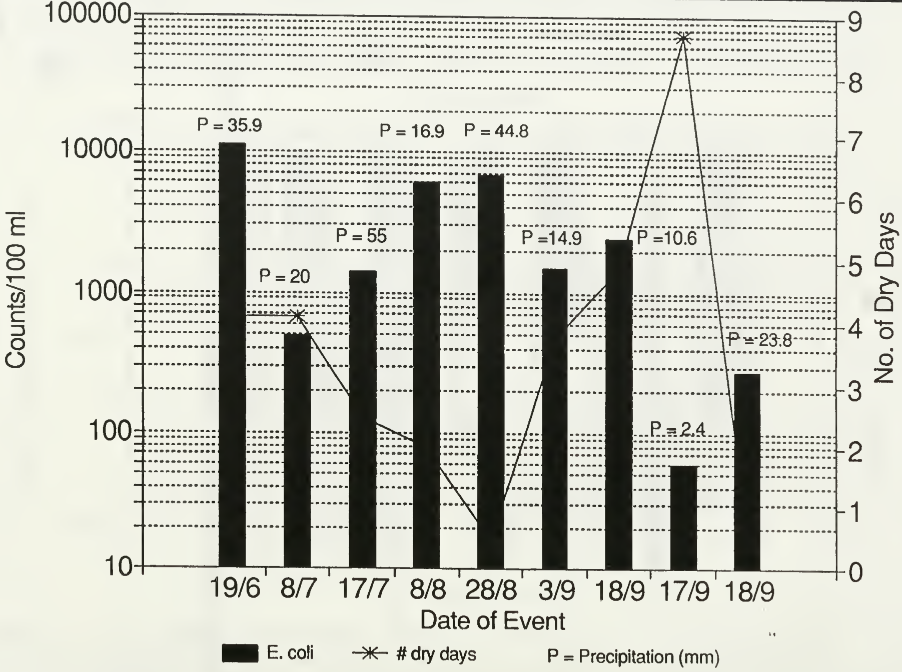

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FIGURE 54. AVERAGED COLIFORM CONCENTRATIONS - HEART'S DESIRE

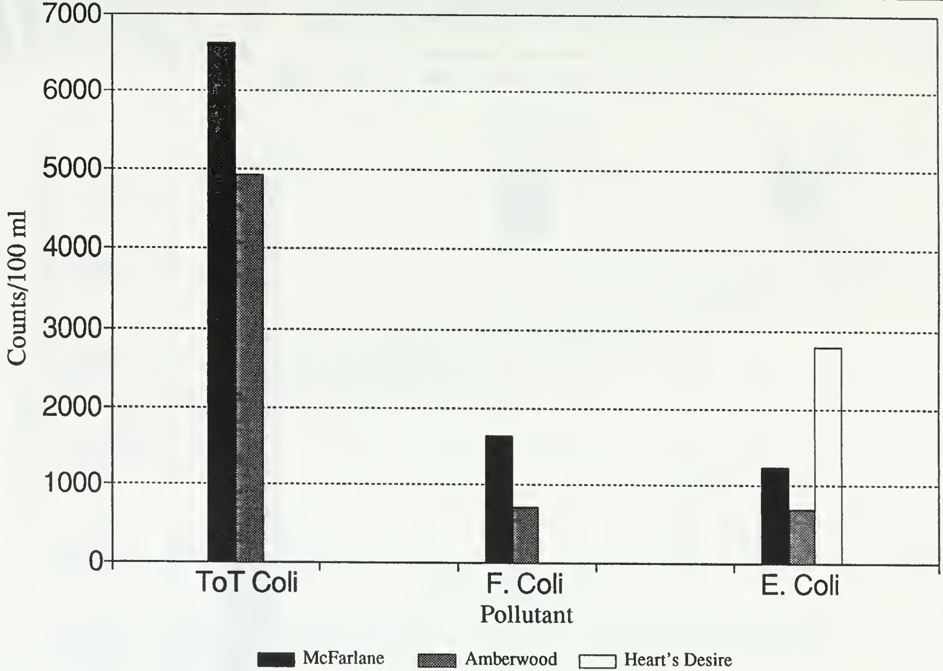

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FIGURE 55. AVERAGED COLIFORM CONCENTRATIONS - 18/6 TO 18/9, BASE LOADING INCLUDED

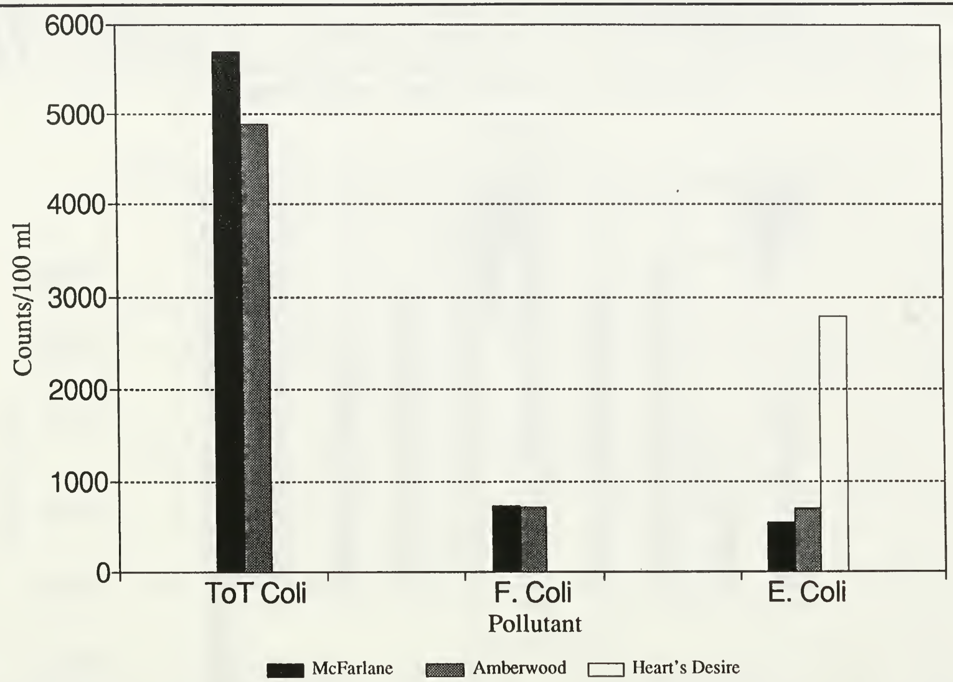

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FIGURE 56. AVERAGED COLIFORM CONCENTRATIONS - 18/6 TO 18/9, BASE LOADING EXCLUDED

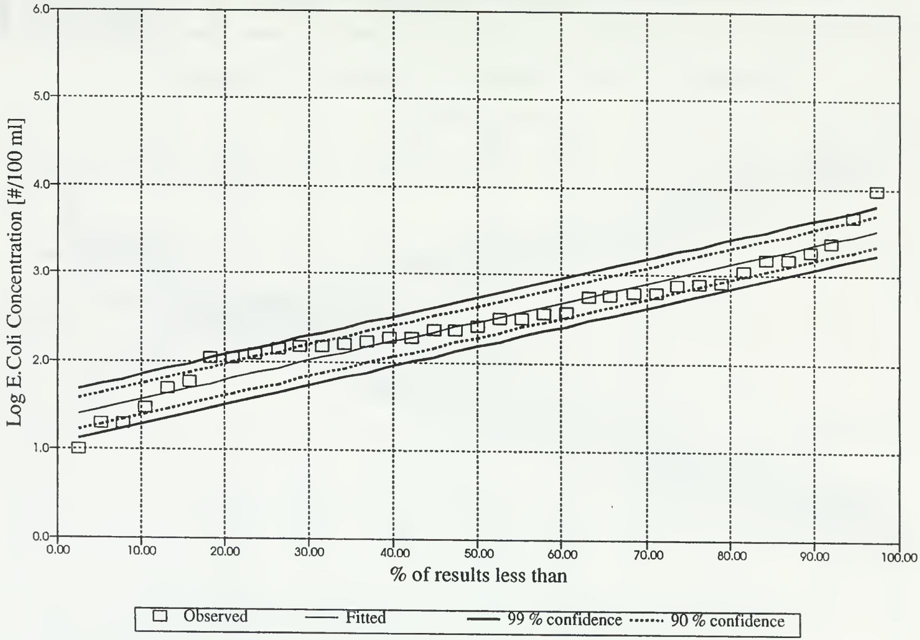

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FIGURE 57: E.COLI CUMULATIVE PERCENTAGE FREQUENCY AT AMBERWOOD

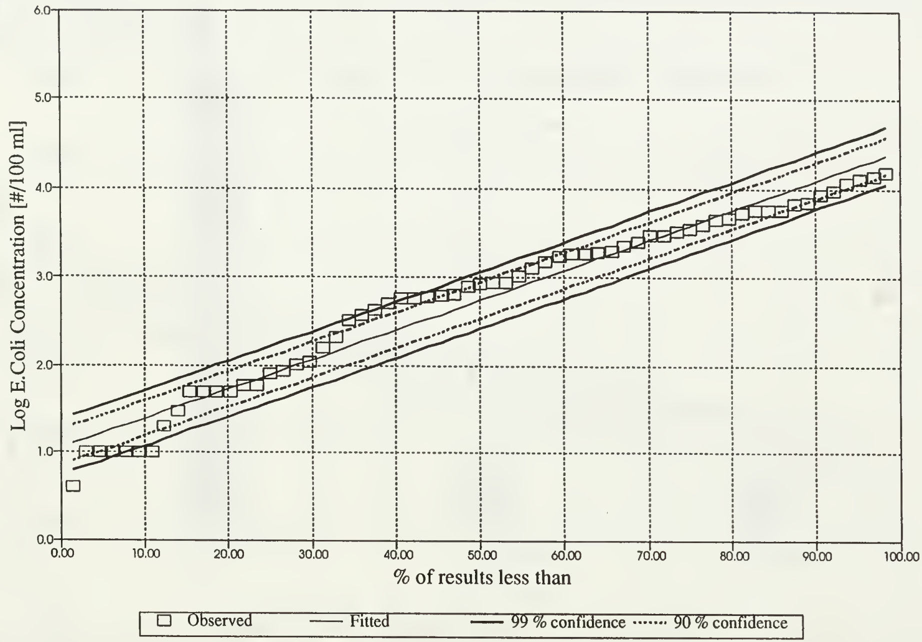

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FIGURE 58: E.COLI CUMULATIVE PERCENTAGE FREQUENCY AT HEART'S DESIRE

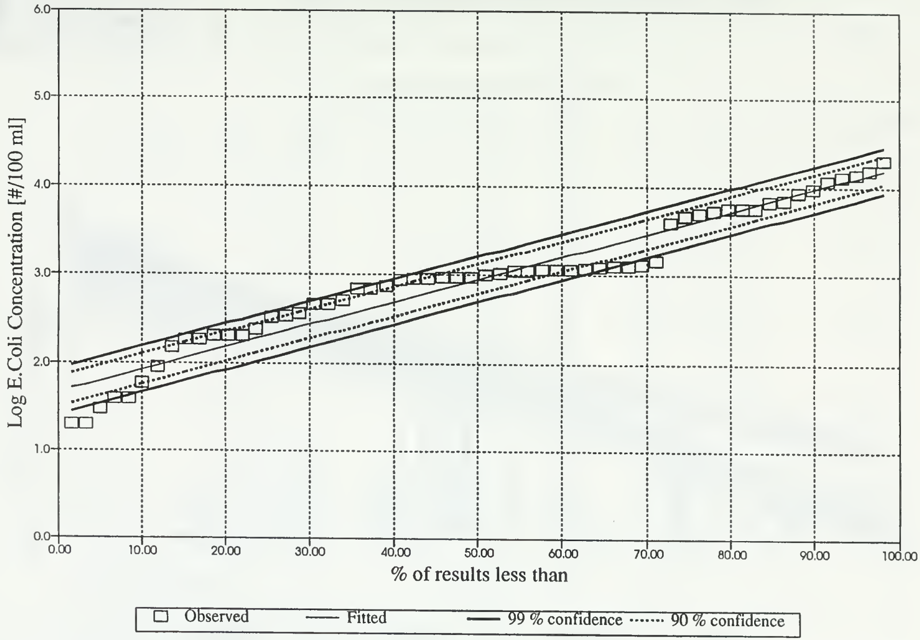

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FIGURE 59. E.COLI CUMULATIVE PERCENTAGE FREQUENCY AT MACFARLANE

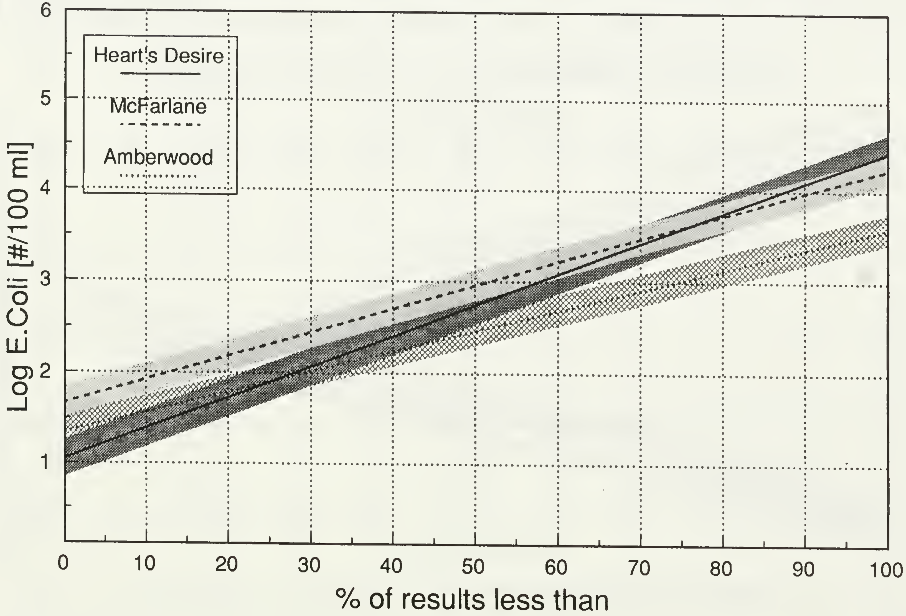

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FIGURE 60: 90% CONFIDENCE INTERVALS (E. COLI)

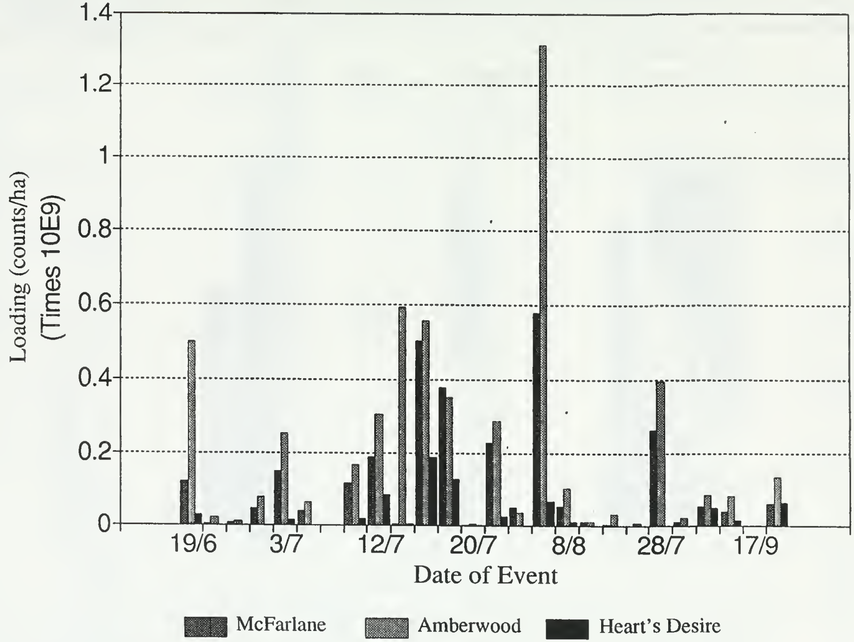

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FIGURE 61: EVENT E. COLI. LOADINGS

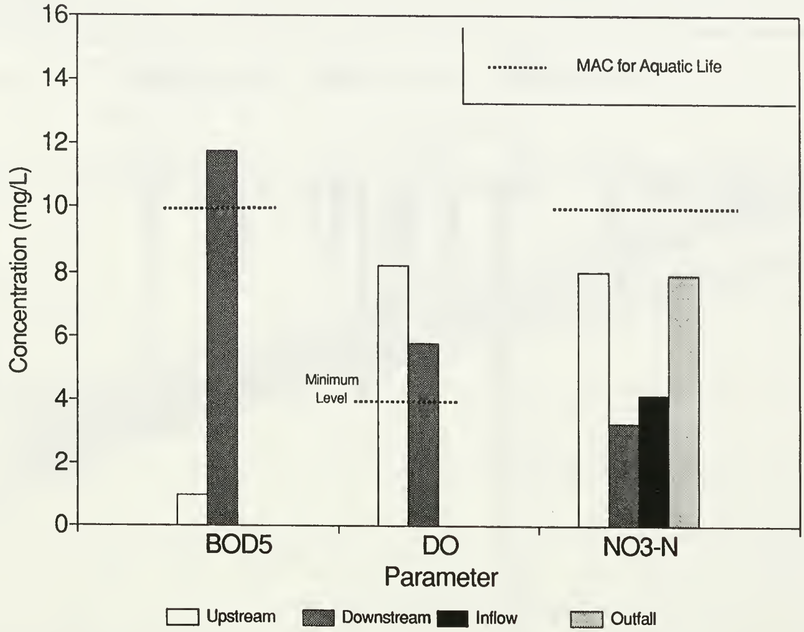

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FIGURE 62: POLLUTANT CONCENTRATIONS IN GROUNDWATER
(EVALUATION OF PERFORATED PIPES)

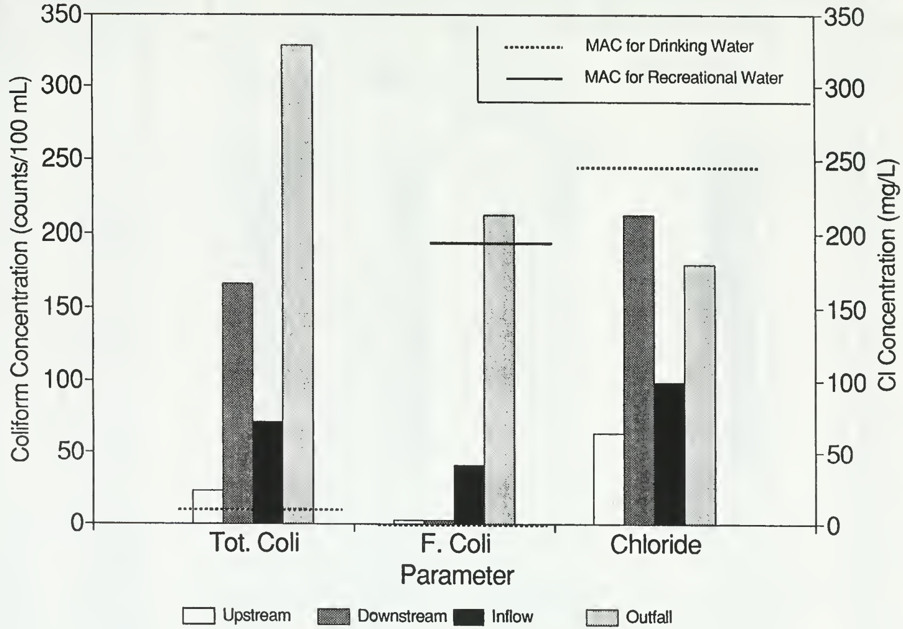

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FIGURE 63: POLLUTANT CONCENTRATIONS IN GROUNDWATER (EVALUATION OF PERFORATED PIPES)

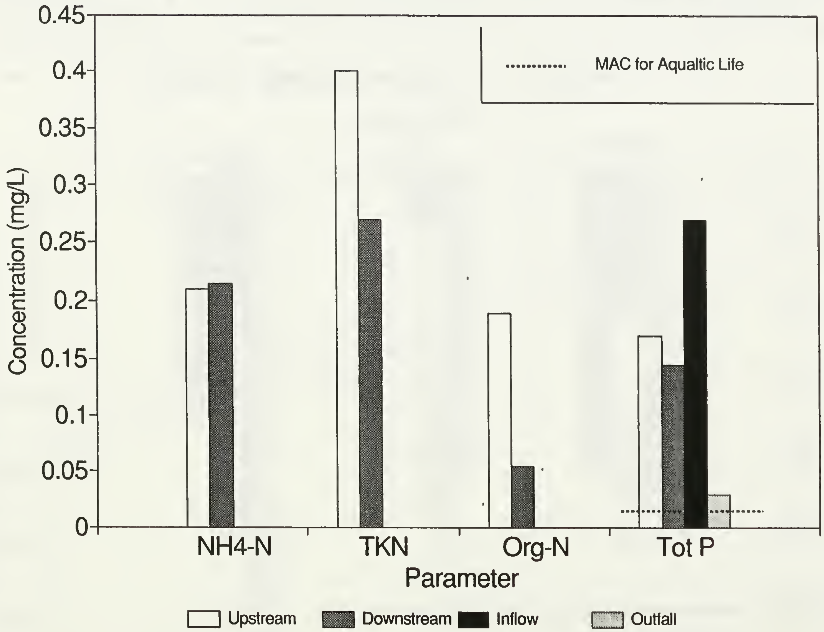

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FIGURE 64: POLLUTANT CONCENTRATIONS IN GROUNDWATER
(EVALUATION OF PERFORATED PIPES)

11.0 GENERAL GUIDELINES AND RECOMMENDATIONS

Based on the tasks undertaken within this study and the findings therein, a number of general guidelines and recommendations can be made for the feasibility, design and construction of grass swale-perforated pipe systems for the purpose of stormwater quality and quantity management.

It is noted that some of the following recommendations are general in nature and their implementations may, in some cases, require the advice of a specialized professional engineer. Furthermore, it is recommendable that the effects and long term efficiency of special design features be investigated through some field monitoring.

Feasibility

- Grass swale-perforated pipe systems should not be considered when native soils are impermeable or when the level of the groundwater table and/or bedrock are near the level of the proposed system. A minimum of 0.6 to 1.2m clearance from the bottom of the pipe trench to the seasonally high water table should be available. The suitability of the system and the minimum clearance should be confirmed by a hydrogeologist.

- The performance of a perforated pipe system installed in an area with accentuated topography may be reduced by the flow which may occur along the gravel bed. In such an area the expected performance of the system may not be achieved unless special design features such as impermeable barriers along the trenches are incorporated. Alternatively, the system can be designed to have horizontal trenches with the use of drop manholes.

- In heavily treed areas, roots may prevent the use of perforated pipes unless special measures such as the use of copper mesh or partly woven fabrics are taken.

Design Considerations

Even if a site is found to be feasible for the use of a grass swale-perforated pipe system, the following considerations should be taken into account in the design process:

- For a conservative pipe sizing, a standard Rational Method type of analysis can be used. However, the design can be optimized if the surface runoff coefficient is based on the directly connected impervious area (rather than the

Design Considerations

Even if a site is found to be feasible for the use of a grass swale-perforated pipe system, the following considerations should be taken into account in the design process:

- For a conservative pipe sizing, a standard Rational Method type of analysis can be used. However, the design can be optimized if the surface runoff coefficient is based on the directly connected impervious area (rather than the total) and if considerations are made to account for swale and trench infiltration rates and available storage. Such considerations can be accounted for with the use of the ANSWAPPS model developed in this study. However, in order to provide access for maintenance, pipe sizes should not be less than 200 mm in diameter.

- The size and distribution of perforations around the circumference of the pipe may play an important role if the system is designed to retain and exfiltrate a significant amount of runoff volume. The effects of the perforations can be investigated with the ANSWAPPS model. However, it was found that a 300 mm (12 in) commercially available pipe with eight 12.5 mm (1/2 in) circular orifices, evenly distributed around the circumference of the pipe at every 50 mm, was more than enough. With some pipe suppliers, the cost for perforations does not change with the number or the size.

- Small system slopes should be emphasized to provide lower flow velocities and to increase flow depths.

- The volume of storage available in the trench should only be based on the portion of the trench which is below the pipe.

- Based on its design volume, the trench should be capable of draining itself within a reasonable time (e.g. 72 hrs) in order to ensure that the trench is empty before the beginning of the next storm.

- If unwanted trench flow is anticipated it can be reduced by installing impermeable membranes or barriers across the trench at catchbasins or manholes.

- To improve the performance of the system, the thickness of the pipe bedding should not only be designed for structural support but also to provide some storage volume. The volume will depend on the design storm used and will vary with the sewershed imperviousness, grass swale characteristics and native soil infiltration capacities. Again the ANSWAPPS model can help with this analysis. As an example, it was found that to retain and exfiltrate the runoff of a 25 mm storm from an area with 25% imperviousness, a bedding depth of 0.6 m by 0.75 m width would be required if the soils beneath the trench are made up of silty loams. Other examples are shown in the table below.

Examples of Granular Bedding Requirements for Perforated Pipe Systems to Retain and Exfiltrate the Runoff of a 25 mm Storm

Effective Imperviousness %	Bedding Depth (m) Type of Native Soils		
	Sand	Loamy Sand	Silty Loam
15	.25	.35	.40
25	.40	.55	.60
35	.55	.70	.80
45	.70	.90	1.00
55	.80	1.00	1.20

Notes:
1) Assumed trench width = 0.75 m
2) Grass swale width = 5.0 m
3) Swale/pipe slope = 1%
4) Pipe diameter = 300 mm
5) Pipe perforations = 8 x 12.7 mm
6) Void ratio of granular material = 0.35
7) Clogging reduction factor = 0.65 (based on design life of 20 years)

- Perforated pipes with filter socks should be installed continuously through catchbasins such as in the City of Nepean design (see Appendix B). This will prevent larger debris from entering the system.

- The catchbasin should not necessarily be located at the lowest point of the grass swale. In order to provide additional surface storage and infiltration, catchbasins can be located up-slope or at an offset from the centre of the swale. Alternatively, the top of the catchbasins can be set at 25-35 mm above the ground surface, but not too high to interfere with lawn maintenance.

- In general, filter socks should always be used with perforated pipes.

- Filter fabric should be placed around the granular trench. In native cohesionless soils such as silt and sand, a non woven fabric, compatible with the soils, will minimize movement of fines into the trench. In root prone areas, partially woven fabric compatible with the soils would increase its strength and would provide greater resistance to tear. The appropriate type of filter fabric should be specified by a geotechnical engineer.

- The use of solid pipes in treed areas may help combat root intrusion.

- Root intrusion in the system can be further minimized with the use of copper or copper sulphate. Delcan (1988) proposed the used of copper wires placed in the trench or copper mesh. Such methods are not compatible with perforated corrugated steel pipes and should therefore only be considered with the use of polyethylene pipes.

- The types of perforated pipes which have been used with grass swales include corrugated steel, corrugated polyethylene and smooth wall polyethylene pipes. The availability of perforated corrugated steel pipes may be limited to small diameters of 200 to 300 mm while the polyethylene pipes are available in much larger sizes.

- In view of the limited depths at which the perforated pipes may be installed, it may be impossible to directly collect flows from foundation drains. In this case, sump pumps may be necessary or a second pipe may be installed deeper, below the perforated pipe.

- In fully serviced areas, sanitary sewer systems and perforated pipes should not be located in the same trench in order to avoid contamination.

- The use of perforated catchbasins and manhole sumps should be investigated.

- The backfill material above the pipe should be well draining except for the top soil layer which should have an adequate depth (150 to 200 mm) and impermeability to maximize short-term surface detention without creating excessive ponding. The top soil should also retain some moisture to prevent the grass from drying out.

- Clear stone of uniform size (e.g. 12-37 mm) should be used in the trench below the pipe.

- The grass swale should be sized and dimensioned to convey or contain the flow or volume of the design storm.

- It was found through hydraulic laboratory tests that the installation of a baffle or weir at the downstream end of a pipe can significantly increase exfiltration rates through the pipe's orifices. However, this will only improve the performance of the system if adequate underground storage is available. Furthermore, this approach may reduce the hydraulic capacity of the pipe required to convey the peak design flow.

Construction and Maintenance Aspects

- From experience and previous field tests, it was found that the performance of a grass swale-perforated pipe system is more likely to succeed when the system is installed within a fully developed subdivision where no other construction activities are present. In previous investigations where the system was in place prior to the housing development, most of the pipes' perforations were fully clogged within two years of their installation. Hence, if a perforated pipe system is considered for a new subdivision, it may be recommended to provide temporary drainage by means of ditches until major construction activities are completed. Such an action would greatly reduce the risk of silt and grit contamination which would render the system totally inefficient.

- During construction, care should be taken to prevent the contamination of the gravel trench from silts and fines. As such, once the gravel has been placed, the installation of the pipe and filter cloth should follow shortly thereafter, and preferably within the same day.

- To further minimize the intrusion of sediments, end pipes should be capped until the installation of the next pipe segment.

- Special measures should be taken during construction to prevent dirt and debris from entering catchbasins and manholes. Such temporary measures should include the use of geotextiles, solid covers or other acceptable methods.

- Backfill material over the trench should be approved by a geotechnical engineer and should drain well to prevent excessive water retention within the swale.

- The final grading should be done by hand and within design specifications.

- The site clean-up should be done before catchbasins and manholes are uncovered.

- System outlets should be built so as to prevent or discourage access by small animals.

- Residents living within areas with grass swale-perforated pipe drainage systems should be informed about the particularities and potential beneficial characteristics of the system. More specifically, and based on observations made during this study, the dumping of grass and leaves in catchbasins should be discouraged. Most importantly, the training of pets to defecate over catchbasins should be prohibited.

- A proper maintenance and monitoring program needs to be derived for grass swale - perforated pipe systems. Such programs should include regular inspections for root intrusion and excessive sediment transport. Grass swales should be maintained in good condition and special care may be required to remove ice or snow at catchbasin locations during the winter - spring season.

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